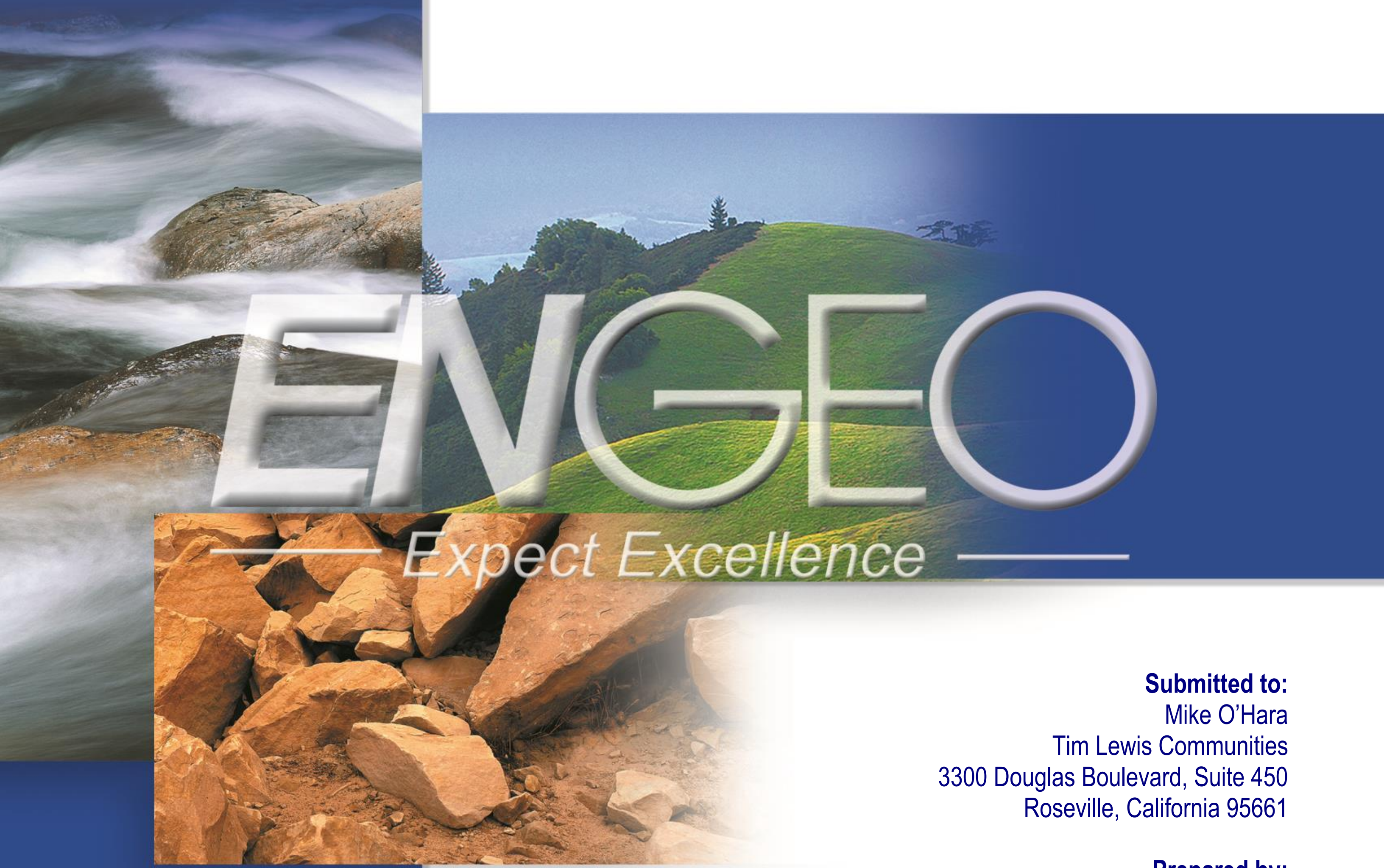
UPDATED GEOTECHNICAL REPORT
ENCINAL TERMINALS
ALAMEDA, CALIFORNIA



Submitted to:
Mike O'Hara
Tim Lewis Communities
3300 Douglas Boulevard, Suite 450
Roseville, California 95661

Prepared by:
ENGEO Incorporated

October 2, 2017

Project No:
9769.000.000

Project No.
9769.000.000

October 2, 2017

Mr. Mike O'Hara
North Waterfront Cove, LLC
c/o Tim Lewis Communities
3300 Douglas Boulevard, Suite 450
Roseville, CA 95661

Subject: Encinal Terminals
Alameda, California

UPDATED GEOTECHNICAL REPORT

Dear Mr. O'Hara:

We prepared this updated geotechnical report for the Encinal Terminals Property as outlined in our agreement dated August 21, 2017. We prepared a preliminary geotechnical report for the project site dated November 19, 2012. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Siobhan O'Reilly-Shah, PE
sors/jf/bvv:gex

Jeff Fippin, GE

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
GEOTECHNICAL
STATE OF CALIFORNIA

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for redevelopment of the Encinal Terminals Property in Alameda, California as outlined in our agreement dated August 21, 2017. The purpose of this report is to evaluate the suitability of the site for the proposed development and to provide geotechnical recommendations for earthwork, foundation design, and groundwater control. You have authorized us to conduct the proposed scope of services, which included the following:

- Service plan development
- Review of previously performed subsurface field exploration
- Review of previously performed soil laboratory testing
- Data analysis and conclusions
- Report preparation

This report was prepared for the exclusive use of North Waterfront Cove, LLC, Tim Lewis Communities, and their consultants for preliminary design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to determine whether modifications are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The project site is located at 1521 Buena Vista Avenue in Alameda, California. The subject property is shown as Assessor's Parcel Numbers (APN) 72-382-1, 72-382-2, 72-383-3, 72-382-9 and a portion of 72-382-10 on the Alameda County Assessor's Parcel Map. The site is about 25 acres of mostly flat land. Figure 1 displays a Site Vicinity Map. The site is bordered to the north by the Oakland-Alameda Estuary, to the west by an inlet known as the Alaska Basin, to the east by the Fortman Marina, and to the south by a warehouse known as the Del Monte Building. A Site Plan is provided as Figure 2.

1.3 PROJECT DESCRIPTION

We understand that the site will be developed with a combination of townhouse and wood-podium multi-family residential structures with associated streets, underground utilities, and landscaping. We understand that engineered cuts and fills for mass grading will be up to 2 feet and 4 feet, respectively. At the time of this report, the final land planning had not yet been completed. The discussion in this report is based on a Conceptual Site Plan prepared by Van Tilburg, Banvard & Soderberch, AA and dated November 26, 2012 and an undated revised land plan illustration provided to us in April 2017.

1.4 SITE BACKGROUND

Existing wharf structures were previously constructed along the western and northern boundaries of the site. The wharves were constructed in three generations and consist of a timber structure in the northwest and two concrete structures along the west. The site was originally developed around the 1920s and was used as a ship berthing and distribution center until the 1980s. The site previously had warehouse buildings along the western portion of the site as well as other smaller buildings and rail spurs throughout the site. In the mid- to late-1980s, the warehouses were removed, and in the 1990s, the site was regraded and repaved to convert the site to a staging area for empty and full shipping containers. It is our understanding that during the repurposing of the site, recycled asphalt was temporarily stockpiled in several areas of the site for several months as a surcharge. We understand that the shipping containers were 40 feet long by 8 feet wide by 8 feet high and were stacked in columns of four containers.

2.0 FINDINGS

2.1 FIELD EXPLORATION

We performed field exploration in three phases:

Our first phase of field exploration included performing five cone penetration tests (CPTs) at the site on November 9, 2012. These CPTs are designated CPT-02-01 through CPT02-05.

Our second phase of field exploration included drilling six borings and advancing five CPTs at various locations on the site. We performed our field exploration from January 17 to 18, 2013, and from January 24 to 28, 2013. The borings are designated B1-1 to B1-5 and the CPTs are designated CPT03-01 through CPT03-05.

Our third phase of field exploration included advancing five additional CPTs to supplement the prior information. We performed the CPTs on July 5, 2013. The CPTs are designated CPT04-01 to CPT04-05.

The locations of our explorations shown on Figure 2 are approximate and were estimated by pacing from points of interest on the site, and the elevations are estimated from regional topographic mapping; they should be considered accurate only to the degree implied by the method used. We permitted and backfilled the explorations in accordance with the requirements of Alameda County Public Works Agency.

2.1.1 Borings

We retained a truck-mounted rig equipped with a 5⅞-inch-diameter mud rotary drill bit to drill six exploratory borings to a maximum depth of approximately 106½ feet below existing grade. Three of the borings were drilled through the concrete wharf structure along the western portion of the site. The borings were logged in the field and soil samples were collected using either a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners, a

2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler or a 3-inch O.D. Shelby Tube sampler. The penetration of the split-spoon samplers was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last one foot of penetration; no conversion factors have been applied. The samplers were driven with a 140-pound hammer falling a distance of 30 inches employing an automatic trip system. We used the field logs to develop the report logs provided in Appendix A.

The boring logs graphically depict the subsurface conditions encountered at the time of the exploration, and they describe the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System (USCS). Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual. Select samples recovered during drilling activities were tested to determine various soil characteristics as described in Section 2.5.

2.1.2 Cone Penetration Tests

We retained a CPT rig to push the cone penetrometer to a maximum depth of about 100 feet. We performed the testing in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix A.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Regional Geology

The San Francisco Bay Valley and the peripheral hill system which encloses it, in association with two main fault structures (the San Andreas and Hayward rift zones), make up the main geological features of the bay region. Diverse crustal movements within this system control the morphology and structural stability of the area.

Because of its close proximity to the Pacific Ocean, the Bay Area's hydrologic, and thus, sedimentologic, conditions are dominated by relative sea level fluctuations and changes in the rate of precipitation. The Bay Area has experienced four episodes of intense erosion followed by four periods of massive deposition in recent geologic history. This process has resulted in the removal of large amounts of bedrock that have been subsequently covered by Pleistocene sediments to considerable depths. We are currently in an interglacial period in which the earth is warming. During this warming period, relative sea level has risen and heavy sedimentation has occurred in the bay valley (the well-documented Young Bay Mud).

The Bay Area can thus be described as a region of depositional and erosional cyclisity with stratigraphic beds that increase in age with depth. The youngest deposits should be expected to be soft and unconsolidated, while the older horizons will be more indurated due to overburden pressure and severe in-situ weathering.

2.2.2 Local Geology

Figure 2 shows the mapped shoreline in 1885. According to Witter (2006), the site is situated in an area mapped as artificial fill over estuarine mud (afem). In general, the stratigraphy of the project site from youngest to oldest consists of (1) artificial fill, (2) Young Bay Mud (YBM) deposits (3) San Antonio Formation and (4) Old Bay Clay (OBC).

As a consequence of the land reclamation in the 1920s, a highly heterogeneous surficial layer of fill material exists on the surface. The fill material is composed of a mixture of sand and gravel. The majority of project site was located in an intertidal marsh area between the historic shoreline and the historic marsh limit. A portion of the site near the northwest corner and the southeast corner are outside of the historic shoreline indicating the area may not have been a marsh. The Alaska Basin, to the west of the site, is in an area where dredging occurred to form most of the basin, according to the historic shoreline map.

The San Antonio Formation underlies the YBM deposits and is sometimes interbedded with the YBM and OBC deposits. This formation is composed of alluvium deposited in environments ranging from alluvial fans and flood plains to lakes and beaches. The unit is generally moderately dense to very dense sand and stiff to hard silt and clay. The Old Bay Clay is characterized by being overconsolidated and fairly stiff due to the overburden of the artificial fill, YBM and San Antonio Formation. The Old Bay Clay is thought to have been deposited during a previous interglacial period (Rogers and Figuers, 1991).

2.2.3 Seismicity

Numerous small earthquakes occur every year in the San Francisco Bay Region and larger earthquakes have been recorded and can be expected to occur in the future. Figure 3 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region. The most common nearby active faults within 25 miles of the site and their estimated maximum earthquake magnitudes (Blake, 2000) are provided in the following table. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart and Bryant, 1997).

TABLE 2.2.3-1
Regional Faults B

Fault Name	Approximate Distance (miles)	Estimate of Maximum Magnitude
Hayward-Rodgers Creek (South & North)	3.8	7.3
Hayward-Rodgers Creek (North)	4.0	7.2
Mount Diablo Thrust	13.6	6.7

ENGEO
Expect Excellence

Fault Name	Approximate Distance (miles)	Estimate of Maximum Magnitude
Calaveras	13.8	7.0
North San Andreas	14.2	8.0
Green Valley Connected	16.8	6.8
San Gregorio Connected	18.7	7.5
Monte Vista-Shannon	23.7	6.5
Greenville Connected	24.2	7.0

The Uniform California Earthquake Rupture Forecast (UCERF3, 2015) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 generated an overall probability of 72 percent for the San Francisco Region as a whole, a probability of 14 percent for the Hayward Fault, 7 percent for the Calaveras fault, and 6 percent for the Northern San Andreas.
The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site; therefore, fault rupture through the site is not anticipated.

The site, as well as the entire island of Alameda, is mapped in a Liquefaction Seismic Hazard Zone in the State of California Seismic Hazard Zones map of the Oakland West Quadrangle (CGS, 2003). This liquefaction susceptibility mapping is based on regional geologic mapping of soil and rock deposits and is not based on site-specific exploration or analyses.

2.3 SUBSURFACE CONDITIONS

The ground surface at the project site consisted of 4 to 10 inches of concrete and asphalt concrete (AC). Based on our exploration data, 6 to 13 feet of artificial fill consisting of medium dense to dense sand and gravel was encountered on the peninsula portion of the project site. Generally, we encountered 10 to 60 feet of YBM beneath the fill material, and interbedded sand, silt and clay of the San Antonio Formation were found underlying the YBM strata. Some of the explorations near the southern extent of the peninsula encountered some clay and silt deposits of the San Antonio Formation interbedded with YBM strata.

The borings drilled from atop the wharf encountered 10 to 20 feet of softer, normally-consolidated material on top of more competent interbedded sand and clay deposits of the San Antonio Formation. The San Antonio Formation extends to a depth of about 60 feet below the wharf deck. Yerba Buena Mud (also commonly called Old Bay Clay) lies beneath the San Antonio formation to a depth of approximately 100 feet.

Isolated layers in the existing fill deposit as well as some of the sand andsilt deposits below YBM in the southern portion of the site are potentially liquefiable. The YBM deposits are highly compressible under loads associated with fill and buildings.

2.4 GROUNDWATER CONDITIONS

We did not observe static or perched groundwater in any of our subsurface explorations due to the types of exploration used. Due to site elevations and proximity to the Oakland-Alameda Estuary and granular nature of the fill, the groundwater level is likely relatively shallow and influenced by tide level. We have assumed the groundwater is approximately 5 feet below existing grade in the analyses performed for the site. During the removal and replacement of existing fills (if performed) as well as most underground construction, temporary dewatering procedures will be necessary to lower the shallow groundwater table so that excavation and working areas are kept reasonably dry during construction. We understand that groundwater and soil contamination is a possibility at this site; therefore, consideration should be given to proper testing and disposal of the water collected from the dewatering process.

2.5 LABORATORY TESTING

We tested select samples recovered during drilling activities to determine various soil characteristics as presented on the following table.

TABLE 2.5-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Unit Weight and Moisture Content	ASTM D-2216 ASTM D-2937	Appendix A
Unconfined Compression	ASTM D-2166	Appendix B
Unconsolidated Undrained Triaxial Compression	ASTM D-2850	Appendix B
Consolidation	ASTM D-2435	Appendix B
Laboratory Miniature Vane Shear	ASTM D-4648	Appendix B

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendix B.

3.0 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for the proposed multi-family residential development. Based on our explorations and review of available published maps and reports for the site, the main geotechnical concerns for the proposed development include: (1) existing non-engineered fill, (2) compressible soil, (3) liquefaction potential, and (4) shallow groundwater. These items and other geotechnical issues are discussed in the following sections of this report and should be considered in the initial planning for the study area.

3.1 EXISTING FILL

Our explorations encountered existing fill of varying thickness. The peninsula portion of the site is underlain by existing fill ranging from 6 to 15 feet below the existing site grades. At the time of this report, no documentation was available indicating that the fill was engineered. Based on our experience and the age of the fill, it is not likely that the fill was placed in an engineered manner, and we recommend that the fill be considered non-engineered.

The presence of non-engineered fill can lead to excessive foundation settlement of structures as well as pavement subgrade instability due to variable soil density and material properties. Once the structures on the site have been demolished, treatment of existing fill typically includes removal and recompaction of soil deemed suitable for reuse. Alternatively, the use of ground improvement, such as rapid impact compaction (RIC), can mitigate the typical risks of non-engineered fill without requiring significant earthwork (removal and replacement) and site dewatering. Buildings founded on deep foundations deriving their capacity below the fill and YBM would remove the need for existing fill mitigation in building areas. Recommendations for improvement of the existing fill are provided in Section 5.1.

3.2 COMPRESSIBLE SOIL

Soft, highly compressible YBM deposits were encountered in explorations at the project site. The location and thickness of these deposits are variable, ranging from 10 to 60 feet in thickness. YBM can settle due to loading from any new fill or structures. The amount of settlement is a factor of proposed loads, thickness of YBM, and previous loads experienced by the YBM deposits. The project site was previously used as a shipping terminal and a staging ground for shipping containers and limited surcharge was performed during transition of the site between the two previous uses as described in Section 1.4. Our laboratory testing indicates that the strength and consolidation state of the YBM are consistent with this use. In general, the upper portions of the YBM are overconsolidated while the lower portions are lightly overconsolidated. We evaluated the settlement potential of the compressible soil with the software program Settle3D Version 2.0 by Rocscience using the Boussinesq analysis method.

The expected settlement due to new building and fill loads at the project site will vary based on the thickness of the YBM. In some of the areas of thicker YBM, mitigation of the compressible soil through surcharging is recommended to reduce excessive settlement. Surcharge fill should be placed above design grade elevations in areas of the site where preconsolidation measures are necessary to reduce settlement. The surcharge fill should remain in place for a period sufficient to allow the desired degree of consolidation to be achieved, such that the risk of settlement is sufficiently reduced for the planned development. At this site, due to the thickness of YBM and anticipated construction schedule, we recommend installing wick drains to speed up the surcharge time. Recommendations for the implementation of a surcharge program are provided in Section 5.2.

In other portions of the site, where the YBM is thinner, based on previous site use, we anticipate that settlement from building loads can be mitigated through use of a stiffened mat foundation designed to address both total and differential settlement due to compression of the underlying YBM.

Utility connections to the buildings will need to be flexible enough to allow for differential movement between the connection to the building and the utility main.

In general, surcharging is not anticipated within the roadway areas, however, depending on the final site plan, fill placement in roadway areas could result in excessive settlement of the roadway surface and underlying utilities. This can be mitigated by surcharging or use of lightweight fill to compensate for the weight of new fill. Where utilities are installed into the YBM layer, the weight of the new fill will be heavier than the YBM removed resulting in settlement of the backfill. Where this occurs, lightweight fill should be used within the backfill to minimize additional loading on the compressible YBM. Recommendations for utility backfill are provided in Section 5.6.1.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include liquefaction, lateral spreading, ground shaking, ground lurching, tsunamis, and flooding. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, landslides or seiches is considered low to negligible at the site.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength because of pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. As previously mentioned, our explorations encountered layers of sand, silty sand and sandy silt that could potentially liquefy under seismic loading.

We performed an evaluation of liquefaction potential on the CPT data with the software program Cliq (version 1.7.1.6) applying the methodology published by Youd et al in 2001. We assumed a groundwater level of 5 feet below existing ground surface, a peak ground acceleration (PGA) of 0.57g, and a moment magnitude (M_w) of 7.1. The PGA value corresponds to the 2016 CBC Maximum Considered Earthquake Geo-Mean Peak Ground Acceleration (PGA_M) as discussed in Section 3.6 of this report. We evaluated the liquefaction potential for the soil encountered below the assumed water table. The results indicate that limited portions of the existing fill material and some of the clayey silt deposits of the San Antonio Formation interbedded with the YBM in the southern portion of the peninsula are potentially liquefiable. The results of the liquefaction analyses are included as Appendix C.

3.3.3.1 Liquefaction-Induced Ground Settlement

We evaluated potential post-liquefaction induced ground settlement at the site using the CPT data and methods outlined in Zhang (2002). Based on our analysis, we estimate that the existing fill and the underlying clayey silt deposits may undergo liquefaction-induced settlement ranging from 0 to 2 inches during a CBC Maximum Considered Earthquake (MCE) seismic event. The settlement at CPT CPT3-01 is an outlier in the data with a settlement of approximately 2 inches while all of the other analyses indicate approximately 1 inch or less of vertical settlement. The resulting differential settlement could be approximately 1 inch over a distance of 50 feet over the majority of the site and 1½ over 50 feet in the southern portion of the site. We propose to mitigate this potential liquefaction settlement though rigid foundation design or with deep foundations. Further discussion of these potential liquefaction mitigation measures are presented in Section 5.1. Recommendations for deep foundations are not provided in this report; however, deep foundation recommendations can be provided if ground improvement measures are not preferred.

3.3.3.2 Lateral Spreading

Lateral spreading is a failure within weak soils, typically due to liquefaction, which causes a soil mass to move toward a free face, such as an open channel, or down a gentle slope. Reduction of the liquefaction risk will reduce the potential for lateral spreading.

The stability of the waterfront due to soft soil failure is discussed under separate cover. If the stability of the northern shoreline is found to have a potential influence on the development, mitigation may include establishing a setback of buildings from the free face, reinforcing existing waterfront structures, and/or performing ground improvement along the project boundary.

3.3.4 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area Region, but based on the site location, it is our opinion that the offset is expected to be nominal.

3.3.5 Tsunami

Due to proximity of the site to the San Francisco Bay, flooding associated with a tsunami is a risk at the site. Tsunami height should be considered in evaluating site design grade.

3.4 SOIL CORROSION POTENTIAL

One soil sample was collected and transported under proper chain-of-custody to CERCO Analytical, Inc. for laboratory testing. Samples were tested for redox potential, pH, resistivity, sulfate ion and chloride ion concentration. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results of each of these tests are summarized below. A detailed description of the laboratory results is contained in the attached report prepared by CERCO Analytical, Inc. (Appendix D).

TABLE 3.4-1
Soil Corrosivity Test Results

Sample Number and Depth	Redox Potential (mV)	pH	Resistivity* (ohms cm)	Sulfate* (mg/kg)	Chloride* (mg/kg)
1-B6 @ 6’	460	8.0	1,200	600	43

*Results reported on a wet weight basis

As indicated in the CERCO laboratory letter (Appendix D), due to the resistivity measurements, the sample was classified as “corrosive”, and buried metal and steel should be protected against corrosion. A corrosion consultant should provide specific design recommendations on corrosion protection for the buried pipeline.

The reported sulfate concentration results were 600 mg/kg, with a detection limit of 15 mg/kg. The 2010 CBC references the ACI (Section 4.3, Table 4.3.2), which provides the following guidelines to characterize the potential exposure for sulfate attack and associated recommendations for concrete in contact with soil based upon the exposure risk.

TABLE 3.4-2
ACI Table 4.3.2

Sulfate Exposure	Sulfate Concentration		Cement Type	Maximum Water/Cement Ratio	Minimum F'_C (psi)
	mg/kg	(%)			
Negligible	0 – 1,000	0.0 – 0.1	---	---	---
Moderate	1,000 – 2,000	0.1 – 0.2	II, IP(MS), IS(MS)	0.50	4,000
Severe	2,000 – 20,000	0.2 – 2.0	V	0.45	4,500
Very Severe	over 20,000	over 2.0	V plus pozzolan	0.45	4,500

In accordance with the criteria presented in ACI Table 4.3.2 table above, the test results are classified in the "negligible" sulfate exposure range. Cement type, water-cement ratio and concrete strength are not specified by the CBC for this range. However, testing was not completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, we can provide additional testing and/or guidance regarding the exposure risk for sulfates. For preliminary planning purposes, we recommend that Type II cement be used in foundation concrete for structures at the project site and concrete should incorporate a maximum water cement ratio of 0.5 and a minimum compressive strength of 3,000 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications, and the final disposition of potential concrete elements is not known at this time.

3.5 EXPANSIVE SOIL

Based on our subsurface exploration, laboratory test results and the preliminary project data presented in Section 1.3, expansive soil should not affect the proposed development. The YBM layer includes highly expansive soil and is not suitable for use as engineered fill on the site.

3.6 2016 CALIFORNIA BUILDING CODE SEISMIC DESIGN PARAMETERS

Based on the subsurface soil conditions encountered and local seismic sources, the site may be designed based on 2016 California Building Code (CBC) seismic design parameters shown in the following table.

TABLE 3.6-1
2016 CBC Seismic Design Parameters

Coefficient	Value
Mapped MCE Spectral Response Acceleration at Short Periods, S_S	1.63
Mapped MCE Spectral Response Acceleration at a Period of 1 second, S_1	0.64
Site Class	E
MCE, 5% Damped, Spectral Response Acceleration at Short Periods Adjusted for Site Class Effects, S_{MS}	1.47

Coefficient	Value
MCE, 5% Damped, Spectral Response Acceleration at a Period of 1 second Adjusted for Site Class Effects, S_{M1}	1.54
Design, 5% Damped, Spectral Response Acceleration at Short Periods, S_{DS}	0.98
Design, 5% Damped, Spectral Response Acceleration at a Period of 1 second, S_{D1}	1.03
Long Period Transition Period, T_L (seconds)	8
MCE Geo-Mean Peak Ground Acceleration, PGA_M (g)	0.57

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. All earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil, rock, and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as determined by an ENGEO representative.

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define "structural areas" as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 EXISTING FILL MITIGATION

As previously discussed, the existing fill is subject to potential settlement under new static loading from new fill and structures due to lack of engineering. We recommend ground improvement of building pads for structures on shallow foundations through densification to mitigate this potential settlement.

We recommend mitigation of the potential settlement through ground improvement methods such as rapid impact compaction (RIC), or approved equal. RIC is a proprietary densification method where a 7- to 8-ton weight is dropped from 3 to 4 feet high on an approximately 5-foot-diameter hammer head. The method has been shown to reduce liquefaction of sandy soil to depths up to 15 feet below ground surface, depending on the fines content of the sand. A less used technology that is locally available is Mammoth Vibro Tamper (MVT). MVT is also a proprietary densification method suitable for mitigation of loose sand deposits to a similar depth as RIC. MVT has been widely used in Japan and is recently available in California but has not been used on as many local projects as RIC. Ground improvement is typically performed by a specialty contractor on a design-build basis.

The ground improvement would have the added benefit of reducing the liquefaction potential of the isolated liquefiable soil in the fill. Our liquefaction analysis indicates that some portions of the site could experience up to 1 inch of total settlement due to liquefaction of silty clay deposits deeper than 15 feet. Due to the deeper depth of these deposits, RIC is not a mitigation option. We propose to mitigate this possible differential settlement by including the differential settlement due to residual liquefaction in the criteria for foundation design. Design criteria for foundations are presented in Section 6.1.

As an alternative to densifing the soil, the existing fill can be removed and recompacted. This operation will require temporary dewatering and drying of soil excavated below the current water table prior to recompacting. Due to contamination onsite and at adjacent properties, the dewatering operation could require treatment and may mobilize contamination on adjacent sites. Because of these considerations, we anticipate that in-situ ground improvement, as discussed in the paragraphs above, will be more cost effective than removal and recompacting the fill.

If the buildings are supported on deep foundations, the need to perform ground improvement to address non-engineered fill would be eliminated.

5.2 SURCHARGE AND WICK DRAINS

Based on currently planned fill thicknesses and the lightly overconsolidated nature of the YBM due to site history, post-construction settlement in streets would be less than 2 inches with differential settlement less than ½ inch over a lateral distance of 50 feet. To reduce post-construction consolidation settlements of buildings on shallow foundations, we recommend "preconsolidation" of the thicker sections of YBM prior to site development with a surcharge program. A surcharge program would involve the placement of temporary fills, uniformly blanketing future building areas until the desired degree of consolidation in these areas has occurred, as determined by a site-specific settlement-monitoring program.

The thickness of required surcharge fill is dependent on the total anticipated areal loads in the building areas, including proposed fill loads and anticipated building loads, the thickness of the compressible material, and the construction schedule. We were provided approximate building loads (dead plus live) for the current development concept in April by FBA Inc., Structural Engineers. We prepared the following table showing our estimated surcharge height for each building; for ease of discussion, we subdivided the site into 4 areas with approximately the same subsurface conditions as shown on Figure 4.

The surcharge program analyzed is a 6-month duration with wick drains spaced at 5 feet on center in a triangular pattern. The thicknesses of surcharge fill and additional engineered fill are provided in the table below. The surcharge fill is the amount of soil that will be put on and removed from the site. The additional fill will be placed and left on the site to accommodate for the settlement caused by surcharging.

TABLE 5.2-1
Additional Fill and Surcharge Fill Thickness

AREA	DESIGN FILL THICKNESS (FEET)	BUILDING	LOAD (PSF)	SURCHARGE FILL THICKNESS (FEET)	ADDITIONAL FILL THICKNESS (FEET)
A	<1	Building B	1,158	14	1
	<1	Building G	425	--	½
	<1	Building H	1,073	13	1
	<1	Building H TH	425	--	½
	<1	Building B	1,158	13	2
	<1	Building I	1,073	47	4½
	<1	Building I TH	425	11	2
	1 to 4	--	--	7	2

To establish a uniform stress distribution in the YBM, the surcharge fill should extend beyond the actual building footprints and site-improvement areas. We recommend that the top of the surcharge be located at least 10 feet horizontally beyond the actual building footprints.

5.2.1 Surcharge Placement and Wick Drain Installation Procedure

Based on our experience, the optimum construction sequence to address the existing fill and compressible soil is as follows:

- Remove and replace or densify existing fills as recommended in Section 5.1. Compact engineered fill in accordance with recommendations in Section 5.7 (below).

- Install vertical wick drains in designated surcharge areas. Wick drains should be placed in a triangular grid pattern no greater than 6 feet on center.

- Wick drains should extend to the dense alluvial deposits and OBC below the YBM.

- Place the recommended thickness of additional engineered fill (including anticipated additional fill to address estimated settlement). Compact engineered fill in accordance with recommendations in Section 5.7.

- Place the recommended thickness of surcharge fill. Compact surcharge fill to at least 85 percent relative compaction.

5.2.2 Surcharge and Settlement Monitoring

We recommended that settlement-monitoring plates be installed prior to surcharge placement to monitor consolidation. We also recommend the installation of vibrating wire piezometers to monitor the pore water pressure dissipation. The number and location of the settlement monitoring plates and the vibrating wire piezometers should be determined by the Geotechnical Engineer when the surcharge staging has been determined. To allow for redundancy, no fewer than two settlement plates should be installed in any surcharge phase. The settlement-monitoring plates should be surveyed to determine elevations at least twice monthly for the first 2 months and once monthly until the Geotechnical Engineer has determined that the desired degree of surcharge driven preconsolidation has been achieved. All readings of settlement should be tied to benchmarks established well beyond the zone of surcharge influence.

5.3 DEEP FOUNDATION DESIGN RECOMMENDATIONS

As an alternative to surcharging, buildings could be supported on driven or auger-cast piles. For our pile recommendations, we analyzed 14-inch and 16-inch-square precast, prestressed concrete piles and 18-inch auger cast piles. The minimum tip embedments, indicated as a depth below finished grade, to achieve a capacity of 100 kips are provided below. We also provide the downdrag load caused by settlement of the Young Bay Mud in the table below.v Charts showing the allowable capacity vs embedment depth are provided as Appendix C.

TABLE 5.3-1
Minimum Tip Embedment

AREA	PILE TYPE	MINIMUM TIP EMBEDMENT (FEET)	DOWNDRAG LOAD (KIPS)
	14-inch Square	78	24
	16-inch Square	75	28
	18-inch Auger Cast	79	24
	14-inch Square	106	109
	16-inch Square	104	125
	18-inch Auger Cast	108	110

C	14-inch Square	92	54
	16-inch Square	89	61
	18-inch Auger Cast	92	54
	14-inch Square	120	120
	16-inch Square	118	137
	18-inch Auger Cast	120	121

If a pile foundation is determined to be feasible for the site, we will refine our capacity estimates and develop lateral load resistance estimates for the preferred pile type.

5.4 GENERAL SITE CLEARING

The contractor should clear areas to be developed of all surface and subsurface deleterious materials, including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. The contractor should clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 5.7. All backfill should be observed and tested by a representative of the Geotechnical Engineer.

5.5 ACCEPTABLE FILL

The onsite existing fill material is suitable as engineered fill material provided it is processed to remove concentrations of plastic clay, organic material, debris, and particles greater than 6 inches in maximum dimension. Because of high plasticity onsite, YBM material is not suitable for use as engineered fill material. Imported fill material should meet the above requirements and have a plasticity index less than 12. The contractor should allow us to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

5.6 UTILITY INSTALLATION

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements.

5.6.1 Soft Soil Settlement

Due to underlying compressible soil, trench backfill could result in settlement if the weight of backfill is greater than the weight of the soil removed during excavation. We recommend that lightweight material be used for at least a portion of the backfill. The preferred alternative for lightweight backfill is controlled density fill with a unit weight between 65 and 90 pounds per cubic foot. As an alternative to controlled density fill, a permeable cellular concrete may be used to compensate for backfill weight. Due to the voids in permeable cellular concrete, buoyancy is not an issue, so lighter weight material can be used reducing the thickness of lightweight material required. To prevent fines migration into the backfill, lightweight aggregate should be avoided, but if used, should be fully encapsulated (top, bottom and sides) with filter fabric.

The thickness of lightweight backfill used should be determined based on two times the thickness of YBM excavated but no less than a minimum thickness of 5 feet in locations where the YBM is encountered. This thickness may be reduced if permeable cellular concrete is used. The required minimum thickness would need to be determined depending on documented unit weight of material as verified by the Geotechnical Engineer during construction.

We recommend using flexible utility connections to accommodate for differential settlement and, where possible, adding additional fall to gravity utilities to accommodate for minor site settlement.

5.6.2 Dewatering

Due to the shallow groundwater table, we anticipate that some excavations at the project site will require temporary dewatering to keep the excavation and working areas reasonably dry during construction. In general, we recommend that excavations should be dewatered such that water levels are maintained no less than 2 feet below the bottom of the excavation during shoring installation and the backfill process. If excessive water collects in the trench, it may be necessary to overexcavate the soft unstable trench soils and replace the soil with free draining rock.

Existing utilities may act as conduit for subsurface water, requiring additional measures for dewatering control. If pipeline construction is performed during rainy months, surface water runoff should be diverted away from the utility excavation.

We understand that groundwater contamination is a possibility at this site, therefore, consideration should be given to proper testing and disposal of the water collected from the dewatering process.

5.6.3 Utility Backfill Placement and Compaction

Soft subgrade conditions should be anticipated to be encountered at the bottom of the excavations. It may become necessary to perform subgrade stabilization to mitigate such conditions. Excavations that bottom in unstable soft soils should be covered with a stabilization fabric overlain by at least 18 inches of aggregate base, subbase or Caltrans Class 1 material. The stabilization fabric shall be Mirafi 600X or an equivalent fabric as approved by the Geotechnical Engineer. Other approaches may be acceptable and we should be consulted if alternative approaches are desired.

Once a suitable firm base is achieved, fills should be placed in thin lifts with the lift thickness not to exceed 12 inches or the depth of penetration of the compaction equipment used, whichever is less. Lightweight equipment should be used when working in soft to medium stiff materials. If lightweight aggregate is utilized, a lightweight vibratory compactor is recommended. Controlled density fill should be placed in lifts deemed thin enough to prevent self-collapse and failure of the fill material.

The contractor should place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches;

2. Moisture condition trench backfill to or slightly above the optimum moisture content. Moisture condition backfill outside the trench;

3. Place fill in loose lifts not exceeding 12 inches; and

4. Compact fill to a minimum of 90 percent relative compaction in structural areas and to a minimum of 85 percent in landscape areas (ASTM D1557).

Fill placed within 6 inches of subgrade level in roadway areas should be compacted to at least 95 percent relative compaction prior to placing aggregate base.

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend for a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

5.7 FILL COMPACTION

5.7.1 Grading in Structural Areas

In structural areas, the contractor should perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches;

2. Moisture condition soil to at least 1 percentage point above the optimum moisture content; **and**

3. Compact the subgrade to at least 90 percent relative compaction. Compact the upper 6-inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade soil has been compacted, the contractor should place and compact acceptable fill (defined in Section 5.5) as follows:

1. Spread fill in loose lifts that do not exceed 12 inches;

2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content; **and**

3. Compact fill to a minimum of 90 percent relative compaction; Compact the upper 6 inches of fill in pavement areas to 95 percent relative compaction prior to aggregate base placement.

Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.7.2 Grading in Landscape Areas

In landscaping areas, the contractor should process, place and compact fill in accordance with Sections 5.6.3 and 5.7.1, except compact to at least 85 percent relative compaction (ASTM D1557).

5.8 SITE DRAINAGE

5.8.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

1. Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage devices.

2. Slope finished grade away from building exteriors at a minimum of 5 percent for a distance of at least 10 feet measured perpendicular to the face of the wall.

3. Discharge roof down spouts into closed conduits and direct away from buildings to appropriate drainage devices.

5.8.2 Subsurface Drainage

Based on our site exploration and current grading concepts for the site, we do not anticipate that subdrainage systems will be necessary. We recommend that we review the site grading plans to further evaluate the need for subdrainage systems as well as observe the earthwork operations during site grading.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results and engineering analysis. As previously discussed, the site has a risk of settlement due to both consolidation of the YBM and liquefaction from a design seismic event. As previously mentioned, we recommend using post-tensioned or conventionally reinforced structural mat foundations to address potential differential settlement. If the design recommendations provided below cannot be achieved by the structural engineer, deep foundations (pile or drilled pier) may be considered to mitigate the potential differential settlement.

6.1 SETTLEMENT CRITERIA

Due to the expected differential settlement at the project site, we recommend using relatively rigid mat foundations, such as post-tensioned, waffle, or conventionally reinforced structural mats. These foundations should be sufficiently stiff to move as rigid units with minimum differential movement. The foundations should be combined with surcharging the building pad in accordance with the recommendations in Section 5.2. After surcharging, the post-construction settlement due to consolidation will be less than 1 inch with a differential settlement less than ¾ inches between columns. The liquefaction settlement (up to 2 inches of total settlement and 1½ inch of differential settlement over a lateral distance of 50 feet) should be added to the consolidation settlement when analyzing the seismic performance of the structure. Our experience indicates a larger amount of architectural distress is commonly allowable due to seismic loads compared to static load performance.

Mats can be designed using an average allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of 2,000 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for load combinations that include wind or seismic.

A modulus of subgrade reaction (k_s) of 80 psi/in can be used for conventionally reinforced structural mat design. Lateral loads may be resisted by friction along the base of the structural mat foundations. We recommend using an allowable coefficient of friction of 0.3 (based on a factor of safety of 1.5).

6.2 SUBGRADE TREATMENT FOR MAT FOUNDATIONS

When buildings are constructed on post-tensioned or conventional reinforced mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745-97 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”.

2. A water-cement ratio of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

7.0 WHARF RECOMMENDATIONS

We understand that the timber wharf structure will be removed while the concrete portion of the wharf will be retrofit (if necessary) and will be potentially integrated into the project as a park or open space. We understand that the concrete portions of the wharf structure are composed of two sections, C-1 and C-2. Section C-1 is estimated to have been built in the 1920s and is supported by 18-inch circular plumb timber piles that have a 30-foot concrete sleeve in the upper portion, and Section C-2 was built in the 1960s and is supported by 18-inch octagonal plumb and batter concrete piles. We understand that the C-2 plumb and batter piles are about 95 feet long and the embedment depth in competent material is approximately 65 feet. We understand that the C-1 piles are estimated to be approximately 80 feet long.

7.1 C-2 BATTER PILE AXIAL CAPACITIES

We drilled three borings from the C-2 wharf structure and performed in-situ SPT tests and various laboratory strength tests to determine the idealized soil profile and the representative soil parameters for the wharf section of the project site. The shear strength data was primarily derived from lab vane shear tests, unconfined compression tests and soil type. Using this data, we determined the axial capacity of the existing C-2 batter piles. A range of the ultimate axial capacities of the C-2 Batter Piles for an embedment depth of 65 feet is provided in the table below. This information supersedes previous information provided to Moffatt & Nichol (the marine structural engineer evaluating the wharves) related to capacity of the C2 batter piles.

TABLE 7.1-1
Ultimate Axial Capacity – C-2 Batter Piles

Tension/ Compression	Ultimate Axial Capacity (kips)
Tension	220 to 310
Compression	270 to 380

8.0 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum concrete flatwork thickness of 5 inches over 4 inches of compacted aggregate base. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

9.0 PAVEMENT DESIGN

9.1 FLEXIBLE PAVEMENTS

The following preliminary pavement sections have been determined for a Traffic Index of 5 to 7, an assumed R-value of 25, and in accordance with the design methods contained in Topic 630 of Caltrans Highway Design Manual.

TABLE 9.1-1
Preliminary Pavement Sections

Traffic Index	AC (inches)	AB (inches)
5.0	2.5	8
6.0	3.0	10
7.0	4	11

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement sections are provided for estimating only. The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

9.2 SUBGRADE AND AGGREGATE BASE COMPACTION

Fill placed within 6 inches of subgrade level in roadway areas should be compacted to at least 95 percent relative compaction prior to placing aggregate base. The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to a minimum of the optimum moisture content prior to compaction. Aggregate Base should meet the requirements for ¾-inch maximum Caltrans Class 2 Aggregate Base in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

9.3 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Encinal Terminals project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify us immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, then notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse that is, reusing without our written authorization. Such authorization is essential because it requires us to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

SELECTED REFERENCES

American Concrete Institute, 2005, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures, ASCE Standard, ASCE/SEI 7-10.

Blake, M.C., Graymer, R.W., Jones, D.L., and Soule, Adam, 2000, Geologic map and map database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California: U.S. Geologic Survey, Miscellaneous Field Studies Map MF-2337, scale 1:75000.

California Building Code (CBC) 2016.

California Department of Transportation, 2006, Highway Design Manual.

California Geological Survey (CGS), 2003, State of California Seismic Hazard Zones, Oakland Weast Quadrangle Official Map, February 14, 2003.

Department of the Navy, Navy Facilities Engineering Command, May 1982, Soil Mechanics, Design Manual 7.1.

Department of the Navy, Navy Facilities Engineering Command, May 1982, Foundation and Earth Structures, Design Manual 7.2.

Division of Mines and Geology, 1997, Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Adopted March 13.

Hart, E.W., and Bryant, W.A., 1997, Fault rupture hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.

Idriss, I.M. and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes; Earthquake Engineering Research Institute.

Ishihara, K. and Yoshimine, M., 1992, Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Japanese Society of Soil Mechanics and Foundation Engineering.

Portland Cement Association, Thickness Design for Concrete Highway and Street Pavements.

Post-Tensioning Institute, 2004, Design of Post-Tensioned Slabs-on-Ground, Third Edition.

SELECTED REFERENCES (Continued)

Radbruch, D.H., 1957, Aerial and Engineering Geology of the Oakland West Quadrangle, U.S. Geological Survey I-239, scale 1:24,000.

Robertson, P. K. and R. G. Campanella, 1988, Guidelines for Geotechnical Design Using CPT and CPTU Data.

Robertson, P.K., 2009, Interpretation of cone penetration tests - a unified approach, Canadian Geotechnical Journal 2009, vol. 46, pp. 1337-1355.

Robertson, P.K., 2009, Performance based earthquake design using the CPT.

Rogers J. D. and Figuers S. H., 1991 Engineering Geology Site Characterization of the Greater Oakland-Alameda Area, Alameda and San Francisco Counties, California

Southern California Earthquake Center (SCEC), 2008, Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Evaluation and Mitigating Seismic Hazards in California, adopted September 11.

Structural Engineers Association of California (SEAOC), 1996, Recommended Lateral Force Requirements and Tentative Commentary.

United States Geological Survey (USGS), 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California

Witter et. al., 2006, Map of Quaternary Geology, Central San Francisco Bay Region, California

Wire Reinforcing Institute, 1996, Design of Slab-On-Grade Foundations, An Update, TF-700-R-03 Update.

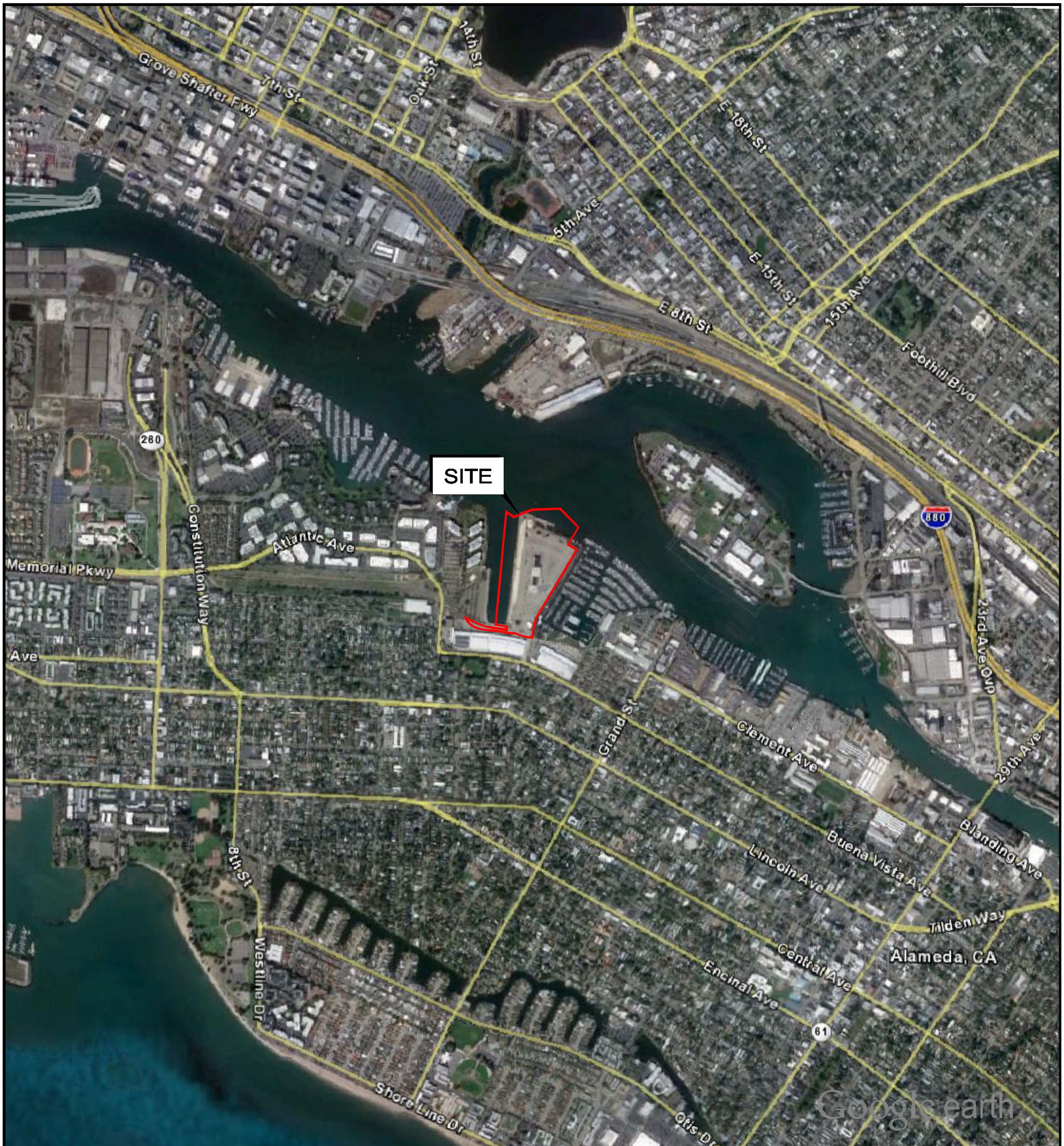
Working Group on California Earthquake Probabilities, 2015, The Uniform California Earthquake Rupture Forecast, UCERF 3, USGS Open File Report 2015-3009.

Zhang, G., Robertson, P.K., Brachman, R. W. I., 2002, Estimating Liquefaction-induced Ground Settlements from Cone Penetration Test for Level Ground, Canadian Geotechnical Journal, Vol. 39, pp. 1168-1180.

Zhang, G., Robertson, P.K., Brachman, R. W. I., 2004, Estimating Liquefaction-induced Lateral Displacements using the Standard Penetration Test or Cone Penetration Test, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 8, pp. 861-871.

FIGURES

F
I
G
U
R
E
S



0 FEET 2000
0 METERS 1000

N

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	VICINITY MAP ENCINAL TERMINALS ALAMEDA, CALIFORNIA	PROJECT NO.: 9769.000.000 SCALE: AS SHOWN DRAWN BY: DLB CHECKED BY: JAF	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg\9769\000\GEX\1017\9769000000-GEX-1-VicMap-1017.dwg Plot Date:10-02-17 spatters

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

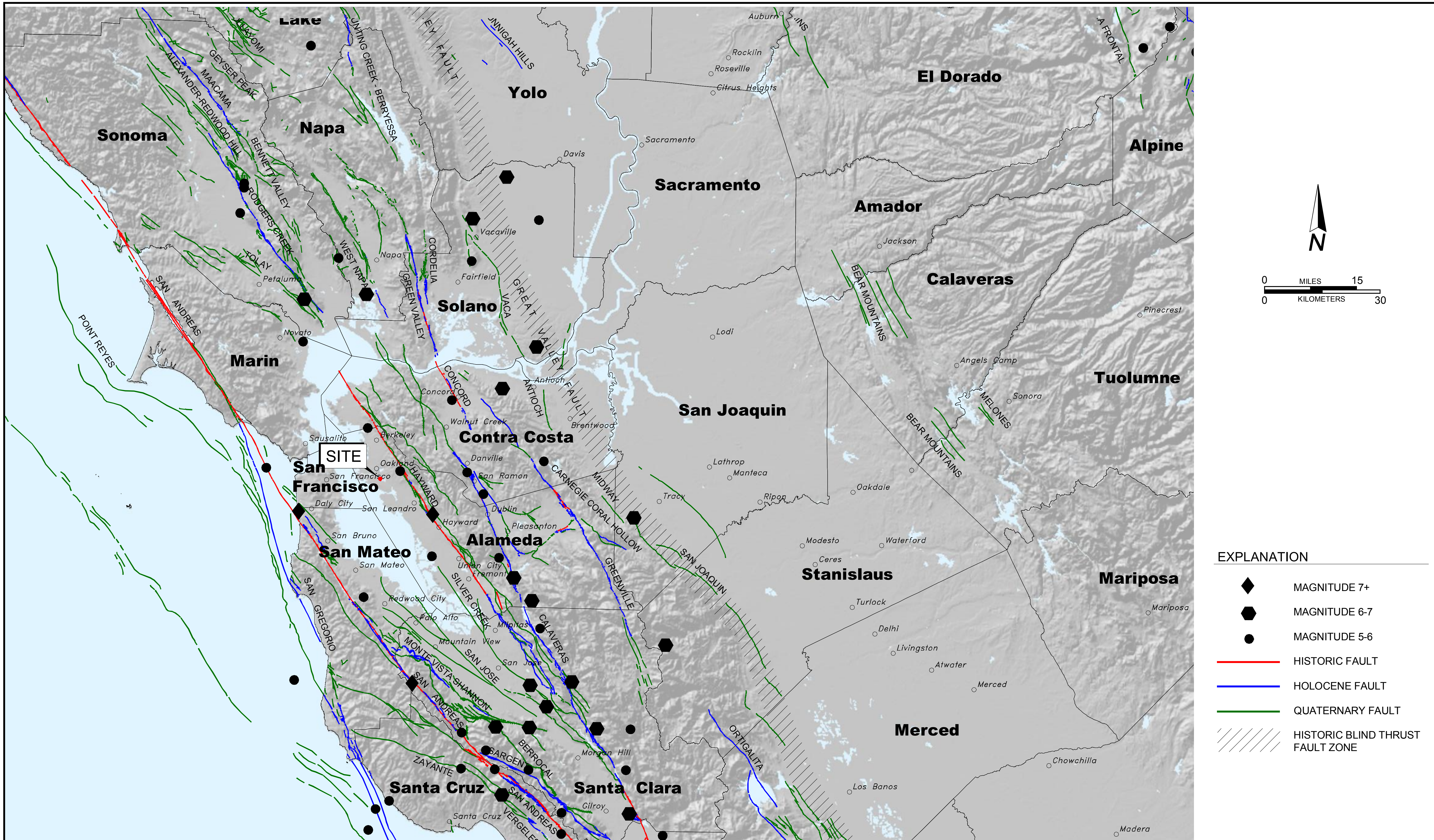
4-CPT05	CONE PENETRATION TEST (ENGEO, 7/2013)
3-CPT05	CONE PENETRATION TEST (ENGEO, 1/2013)
2-CPT05	CONE PENETRATION TEST (ENGEO, 11/2012)
B1-6	BORING (ENGEO, 1/2013)
— — —	LOCATION OF 1885 SHORELINE

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	SITE PLAN ENCINAL TERMINALS ALAMEDA, CALIFORNIA	PROJECT NO.: 9769.000.000 SCALE: AS SHOWN DRAWN BY: DLB CHECKED BY: JF	FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg\9769\000\GEX\1017\9769000000-GEX-2-SitePlan-2-1017.dwg Plot Date:10-02-17 spatters



BASE MAP SOURCE:
U.S.G.S. 1-ARC SECOND S.R.T.M. DATABASE
U.S.G.S. QUATERNARY FAULT DATABASE, MARCH, 2006
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY ENCINAL TERMINALS ALAMEDA, CALIFORNIA	PROJECT NO.: 9769.000.000		FIGURE NO. 3
	SCALE: AS SHOWN		
	DRAWN BY: DLB	CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg\9769\000\GEX\1017\9769000000-GEX-3-RegFltMap-1017.dwg Plot Date:10-02-17 spatters

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	CONCEPTUAL SURCHARGE AND SETTLEMENT PLAN	PROJECT NO.: 9769.000.000	FIGURE NO.
	ENCINAL TERMINALS	SCALE: AS SHOWN	4
	ALAMEDA, CALIFORNIA	DRAWN BY: SOG / CHECKED BY: JAF	

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

APPENDIX A

Key to Boring Logs
Exploration Logs

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4, 3", 12

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
Expect Excellence

LOG OF BORING B1-1

ENGEO — Expect Excellence

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/25/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77853	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25965	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		9 inches concrete, wharf deck								
19		Mud line, drill casing to 28 feet								
20		SANDY CLAY (CL), dark brown, very soft, wet, logged from cuttings.								
25										

ENGEO
Expect Excellence

LOG OF BORING B1-1

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/25/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77853	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25965	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
25–29										
29		WELL-GRADED SAND (SW), brown, medium dense, wet, medium-grained. (San Antonio Formation)								
30–31.5					48	19.9	112			
35–36.5		Dense, fine-grained, with silt.			31					
40–41.5		Very dense.			87					
44		LEAN CLAY (CL), gray mottled with brown, very stiff, moist, <5% very fine sand. (Old Bay Mud)								
45–46.5					26	22.1	109.4	2007		TxUU
47–50										

LOG OF BORING B1-1

ENGEO — Expect Excellence

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/25/2013
Hole Depth (ft)	: 106.5
Surface Elev (ft-msl)	: 7
Latitude (NAD83)	: 37.77853
Longitude (NAD83)	: -122.25965
Logged/Reviewed By	: J. White/ J. Fippin
Drilling Contractor	: Gregg Drilling
Drilling Method	: Mud Rotary
Hammer Type	: 140lb Auto
Hole Diameter (in)	: 5 7/8

Depth in Feet	DESCRIPTION	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
50.5	CLAYEY SAND (SC), grayish brown, dense, wet, fine-grained sand. (San Antonio Formation)	37	16.6	118.6			
55.2	SILTY SAND (SM), brown, very dense, wet, very fine-to fine-grained. (San Antonio Formation)	68					
59	SANDY CLAY (CL), gray, very stiff, wet, few fine gravels, very fine-grained sand. (Old Bay Mud)						
61		28				*4.0	PP
62.5	LEAN CLAY (CL), gray mottled with light brown, very stiff, moist, shells in cuttings at 62 feet, <5% very fine-grained sand and gravel. (Old Bay Mud)						
65.5		20	44.3	78.5	1426		TxUU
70.5		14	59.4	64.9		*2.5	PP
71.5	Decreasing sand and gravel, abundant shells.						

ENGEO
Expect Excellence

LOG OF BORING B1-1

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/25/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77853	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25965	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
75–76.5		Abundant shells.			19				*2.5	PP
80–81.5					12				*2.0	PP
85–86.5		Dark gray, very stiff, decreasing shell content.			28				*2.5	PP
90–91.5		With organics, stiff.			31				*2.0	PP
95–96.5		Increasing sand content.			62				*3.5	PP
96		SANDY CLAY (CL), dark gray, very stiff, moist, very fine-grained sand and shells. (Old Bay Mud)								
100										

03-22-2013 G:\Active Projects\9769\GEX\Exporation\January 2013\Boring Logs\9769 borelogs B1-1.bor

LOG OF BORING B1-1

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/25/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77853	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25965	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
100	■									
101	■				63	19.2	113.5	4057		TxUU
102		Becomes gray mottled with light brown, hard, very fine grained sand.								
103										
104										
105	■									
106	■	CLAYEY SAND (SC), brown and gray, dense, wet, fine grained sand.			48					
107		Bottom of boring at 106.5 feet.								
108										
109										
110										
111										
112										
113										
114										
115										
116										
117										
118										
119										
120										
121										
122										
123										
124										
125										

LOG OF BORING B1-2

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/24/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77917	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25956	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		10 inches concrete, wharf deck								
18		Mud line, drill casing to 31 feet								
19		FAT CLAY (CH), very dark gray, very soft, wet, logged from cuttings.								

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/24/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77917	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25956	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
27		WELL-GRADED SAND (SW), brown, wet, fine grained, logged from cuttings.								
32		FAT CLAY (CH), greenish gray, soft, wet, logged from cuttings.								
35					17	19	113.8			
36		SILTY SAND (SM), greenish gray, medium dense to very dense, wet, fine-grained, becomes brown at 36.5 feet. (San Antonio Formation)								
40					60					
41.5		Brown, very dense, wet, fine-grained.								
45		SANDY CLAY (CL), greenish gray, stiff, wet, fine-grained sand, few shell fragments. (Old Bay Mud)			19				*2.0	PP

LOG OF BORING B1-2

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/24/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77917	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25956	Hole Diameter (in)	: 5 7/8

Depth in Feet	DESCRIPTION	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
50–51		40	27.3	121.2	3196		TxUU
51	CLAYEY SAND (SC), grayish green, dense, wet, fine-grained. (San Antonio Formation)						
54	LEAN CLAY (CL), brown mottled with greenish gray, stiff to hard, wet, few shell fragments, very fine-grained sand. (Old Bay Mud)						
55.5–56.5		20				*2.0	PP
55.75	SILTY SAND (SM), greenish gray, medium dense, wet, fine-to medium-grained sand. (San Antonio Formation)						
59.5	LEAN CLAY (CL), grayish geen, stiff to hard, moist, few shell fragments, very fine-grained sand. (Old Bay Mud)						
60–61.5		34	22.6	104.9	3486		TxUU
65–66.5		31				*4.5	PP
66.5	Hard.						
68	Increasing shell content in cuttings.						
70–71.5		11	40.1	81.3		*2.5	PP
71.5	Very stiff, with abundant shells.						

LOG OF BORING B1-2

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/24/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77917	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25956	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
75–76.5					18				*2.5	PP
80–81.5		Becomes gray, very stiff.			23				*3.0	PP
83		Abundant shells in cuttings from 83 to 84 feet.								
85–86.5		Few organics			23				*3.5	PP
90–91.5		Becomes brown, abundant organics.			32	33.4	87.9	1976		TxUU
95–96.5		Very fine-grained sand, increasing shell content, few fine gravels.			36				*3.0	PP
99		SILT (ML), greenish gray, very stiff, wet, with clay, <5% very fine grained sand, low plasticity. (San Antonio Formation)								

LOG OF BORING B1-2

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/24/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77917	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25956	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
100–101.5					43	23.5		3056		TxUU
105–106.5		SILTY SAND (SM), gray, dense, wet, fine-grained sand.			45				*3.0	PP
106.5		Bottom of boring at 106.5 feet.								

LOG OF BORING B1-3

ENGEO — Expect Excellence

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/28/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78003	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25945	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		10 inches concrete, wharf deck								
19		Mud line, drill casing to 40 feet								
20		WELL-GRADED SAND (SW), brown, logged from cuttings.								

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

LOG OF BORING B1-3

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/28/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78003	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25945	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
25–28.5										
28.5		FAT CLAY (CH), greenish gray, fine-grained sand, logged from cuttings.								
36.5		SILTY SAND (SM), dark grayish green, fine-grained sand, logged from cuttings. (San Antonio Formation)								
40		Brown, dense, wet, fine-grained sand.			55	20	112.2			
44.8		LEAN CLAY (CL), greenish gray, stiff, wet, fine grained sand. (Old Bay Mud)			16	17.8	116.6		*2.0	
50										

LOG OF BORING B1-3

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/28/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78003	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25945	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
50–51.5	■	With silt and carbonates.	▨		38	22	107.9	3706		TxUU
55–56.5	■	SILTY SAND (SM), brownish gray, dense, wet, fine-to medium-grained. (San Antonio Formation)	🟩		81					
60–61.5	■	LEAN CLAY (CL), greenish gray, very stiff, wet, carbonates, fine-grained sand, abundant shells. (Old Bay Mud)	▨		40					
62		stiff	▨							
65–66.5	■		▨		12					
70–71.5	■		▨		9					
75										

ENGEO
Expect Excellence

LOG OF BORING B1-3

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/28/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78003	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25945	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
75–76.5					7	58.4	66.6	523		TxUU
80–81.5					13					
85–86.5					15					
89.5		LEAN CLAY (CL), gray, very stiff, wet, organics, carbonates. (Old Bay Mud)								
90–91.5					43	21.6	108.9	3494		TxUU
95–96.5					46					
96.5		Brown, fine-grained sand, few gravels.								

ENGEO
Expect Excellence

LOG OF BORING B1-3

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/28/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 106.5	Drilling Contractor	: Gregg Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78003	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25945	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
100										
101					71					
102										
103		SILTY SAND (SM), gray, dense, wet, fine-grained sand.								
104										
105										
106					38					
107		Bottom of boring at 106.5 feet.								
108										
109										
110										
111										
112										
113										
114										
115										
116										
117										
118										
119										
120										
121										
122										
123										
124										
125										

ENGEO
Expect Excellence

LOG OF BORING B1-4

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/17/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 42	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77872	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25828	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		4 inches Asphalt, 8 inches Aggregate Base Rock.								
1		CLAYEY GRAVEL (GC), brown, medium dense, moist, fine to coarse gravel. (Fill)								
6					38					
6.5		SILTY SAND (SM), graysih brown, loose, wet, fine to coarse grained sand. (Fill)								
11					4					
11.5		FAT CLAY (CH), gray, very soft, wet. (Young Bay Mud)								
16								*300		TV
21								*300		TV
22		Dark gray, very soft.								
25										

LOG OF BORING B1-4

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/17/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 42	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.77872	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25828	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
25–27						17.8	116.5	2798		TxUU
27		Very fine sand, very dark gray.								
30–32								430		VS
32		Very dark gray								
35–37						116.5	39.8	72 347		TxUU VS
37		Becomes dark brownish gray, soft, few organics								
41–42		LEAN CLAY (CL), light brown, stiff, <5% fine grained sand, abundant organics. (San Antonio Formation)				226.3	24	833		TxUU
42		Bottom of boring at 42 feet								

ENGEO
Expect Excellence

LOG OF BORING B1-5

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/17/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 68.5 feet	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	:	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78145	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25770	Hole Diameter	: 4 inches

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		4 inches Asphalt, 8 inches Aggregate Base Rock.								
1		SILTY SAND (SM), brown, medium dense, wet, fine grained, few gravels. (Fill)								
2										
3					26					
4		SILTY GRAVEL (GM), reddish brown mixed with brown, dense, wet, with fine grained sand, 1/4 inch to 1/2 inch gravel. (Fill)								
5										
6					52	10.3	126.3			
7										
8		WELL-GRADED SAND (SW), dark gray, medium dense, wet, fine-to medium-grained. (Fill)								
9										
10										
11					27					
12										
13										
14		FAT CLAY (CH), dark greenish gray mottled with black, stiff, wet, few organics. (Young Bay Mud)								
15										
16						65.9	59.9	1228 1966		TxUU VS
17										
18										
19										
20										
21								757		VS
22		Dark gray, soft, with organics, some black staining.								
23										
24										
25										

ENGEO
Expect Excellence

LOG OF BORING B1-5

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/17/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 68.5 feet	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	:	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78145	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25770	Hole Diameter	: 4 inches

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
25										
26						96.2	46.2	*400		TV
27		Dark gray.								
28										
29										
30										
31								551		VS
32										
33										
34										
35										
36								593		VS
37		Dark greenish gray, very soft, no organics.								
38										
39										
40										
41								520		VS
42		Few organics, some water intrusion.								
43										
44										
45										
46								418		VS
47		Very soft.								
48										
49										
50										

ENGEO
Expect Excellence

LOG OF BORING B1-5

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/17/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 68.5 feet	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	:	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78145	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25770	Hole Diameter	: 4 inches

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
50										
51								*150		TV
52										
53										
54										
55										
56						94.2	46.5	66		VS
57										
58										
59										
60										
61										
62		No recovery.								
63		Dark gray, very soft.			26			*240		TV
64										
65		LEAN CLAY (CL), dark grayish green, stiff, wet, with fine grained sand. (Old Bay Mud)								
66										
67		No recovery, drilled out to 67, then drive sample.								
68					10	34.4	88.7		*1.25	PP
69		Bottom of boring at 68.5 feet								
70										
71										
72										
73										
74										
75										

ENGEO
Expect Excellence

LOG OF BORING B1-6

Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/18/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 32	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78023	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25826	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
0		3 inches Asphalt, 6 inches Aggregate Base Rock.								
1		WELL-GRADED GRAVEL (GW), brown, medium dense, wet, fine to coarse gravel, with sand and silt. (Fill)								
6					44					
6.5		WELL-GRADED SAND (SW), dark gray, medium dense, wet, fine-to coarse-grained. (Fill)								
8.5		FAT CLAY (CH), dark gray, soft, wet, fine grained sand. (Young Bay Mud)								
11.5						52.5	67.4	661		TxUU
16.5						110.4	41.9	854		VS
17		Gray, medium stiff.								
21.5								607		VS
22		Soft, few shell fragments.								

LOG OF BORING B1-6

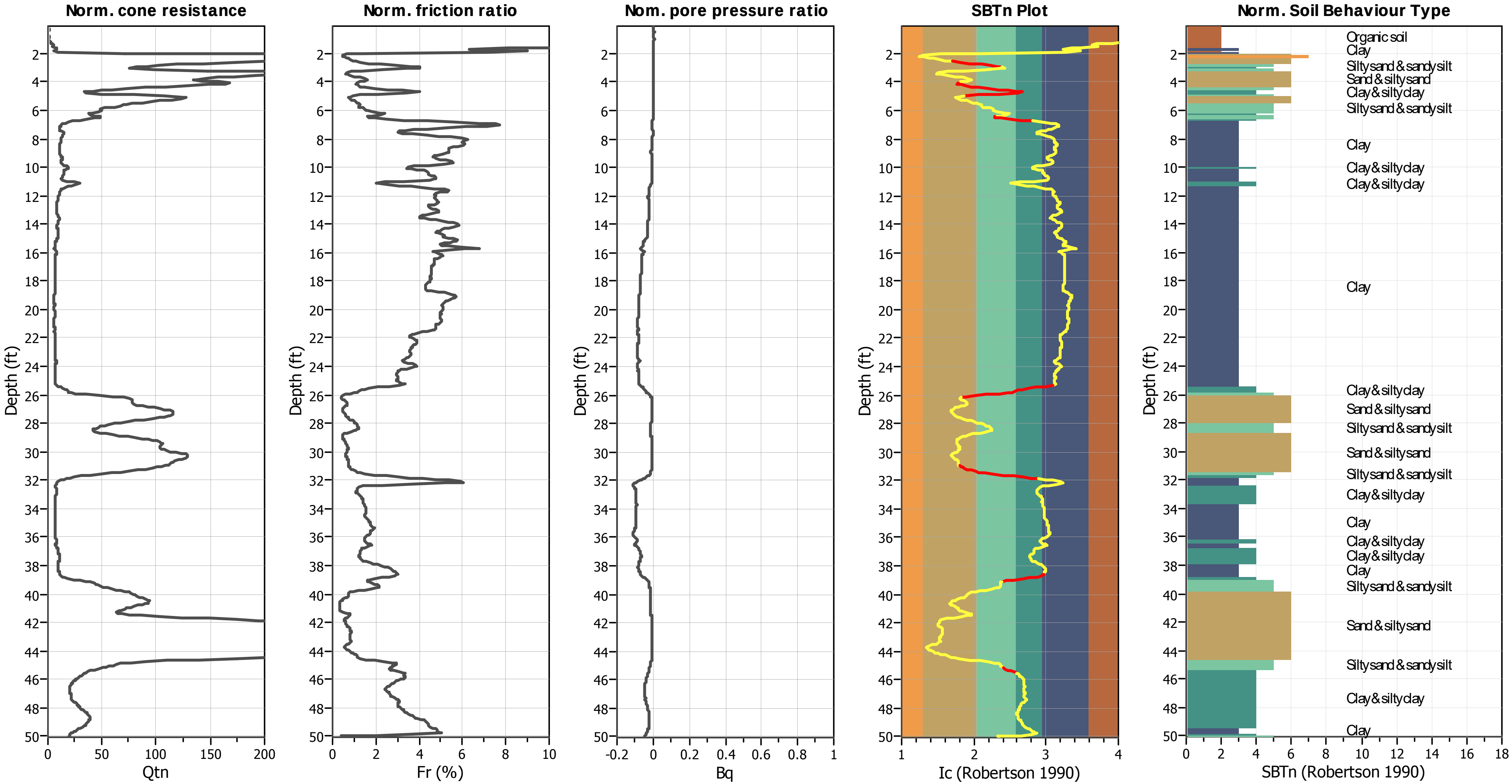
Encinal Terminals
Alameda, CA
9769.000.000

Date Drilled	: 1/18/2013	Logged/Reviewed By	: J. White/ J. Fippin
Hole Depth (ft)	: 32	Drilling Contractor	: V&W Drilling
Surface Elev (ft-msl)	: 7	Drilling Method	: Mud Rotary
Latitude (NAD83)	: 37.78023	Hammer Type	: 140lb Auto
Longitude (NAD83)	: -122.25826	Hole Diameter (in)	: 5 7/8

Depth in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approx	Unconfined Strength (tsf) *field approx	Test Type
26.5						32.3	86.4	135		TxUU
27		Very soft.								
30						*400				TV
30.5		SANDY CLAY (CL), brown, stiff, wet, fine-to medium-grained sand. (San Antonio Formation)								
31.5									*2.0	PP
32		Bottom of boring at 32 feet								

03-22-2013 G:\Active Projects\9769\GEX\Exporation\January 2013\Boring Logs\9769 borelogs B1-6.bor

CPT basic interpretation plots (normalized)



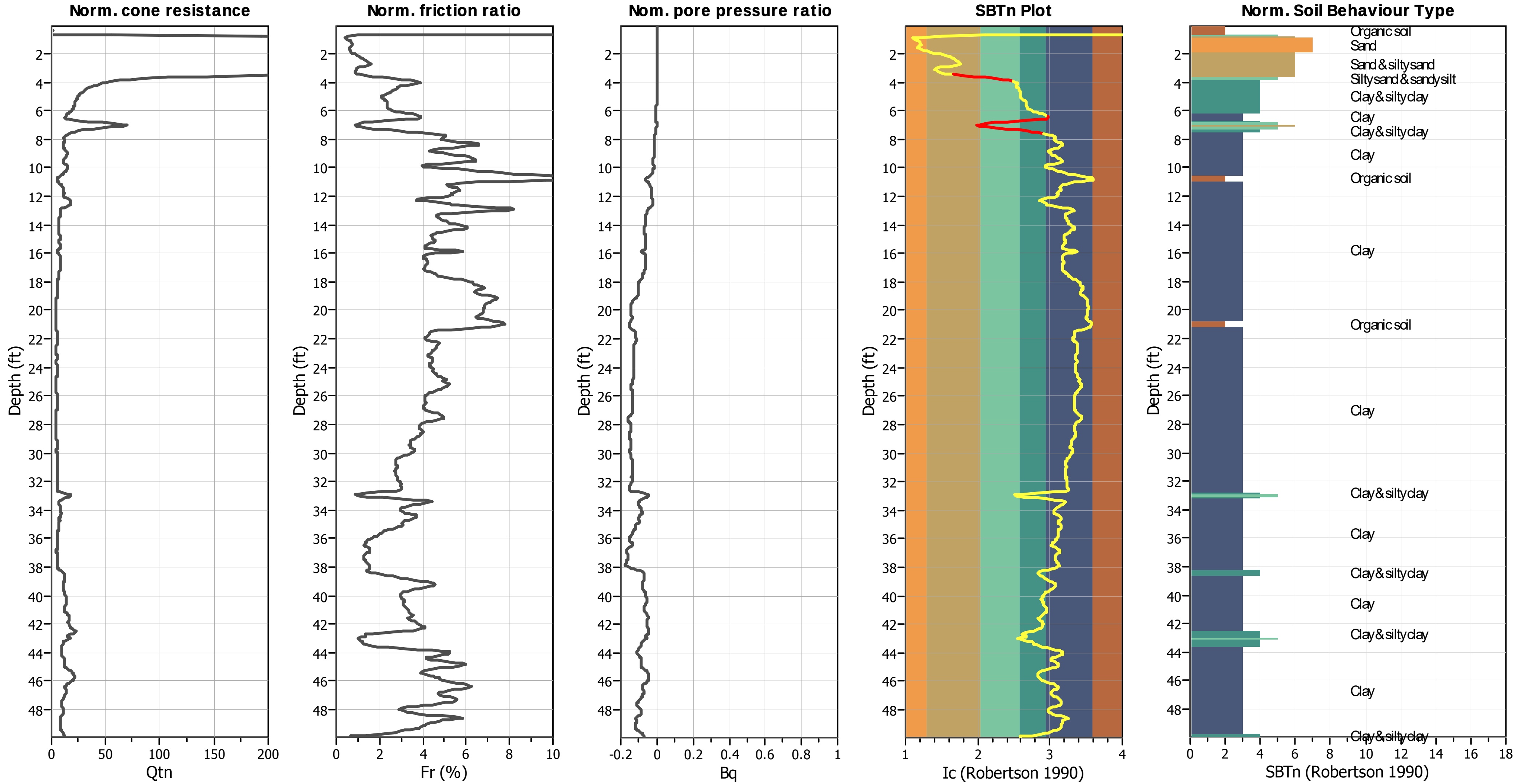
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



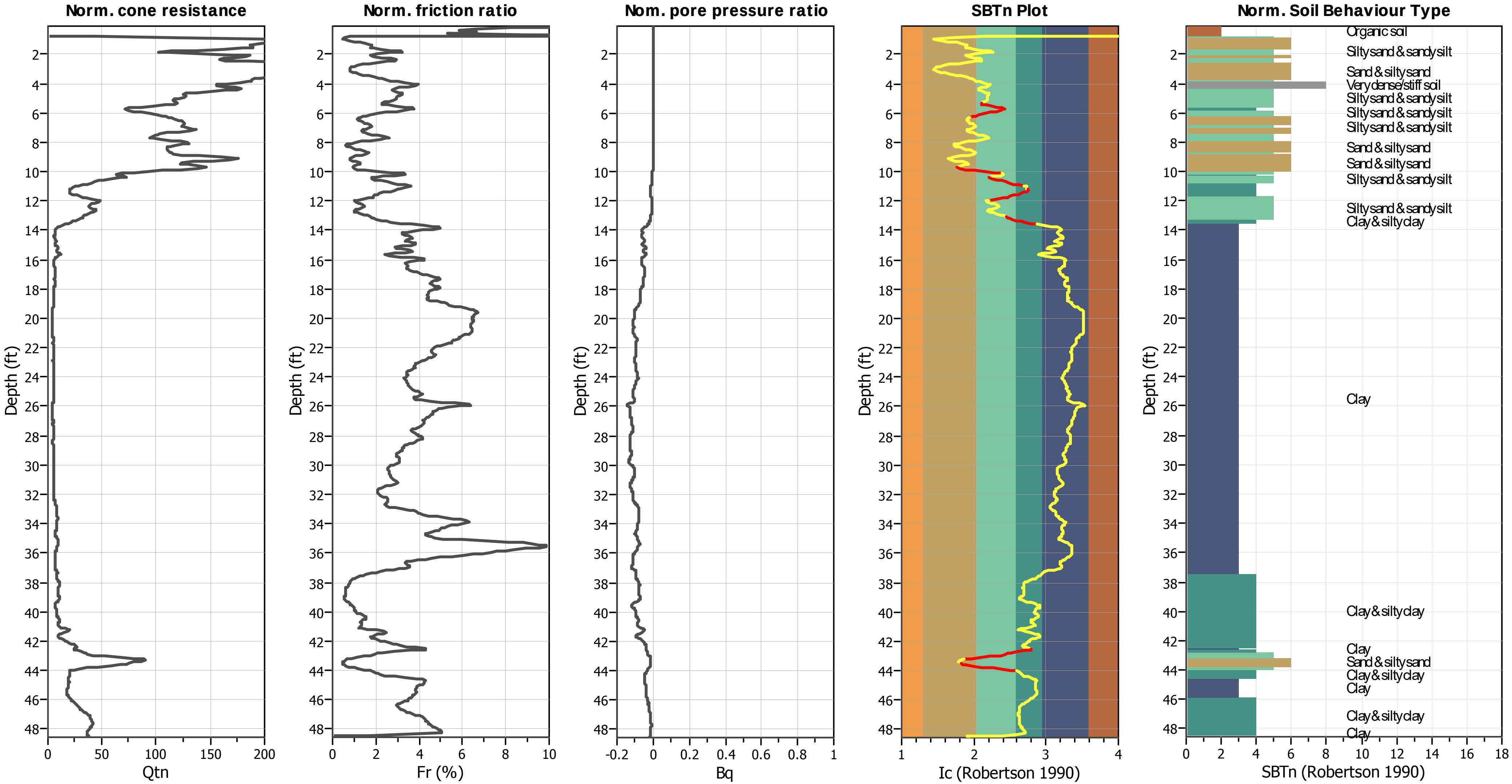
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



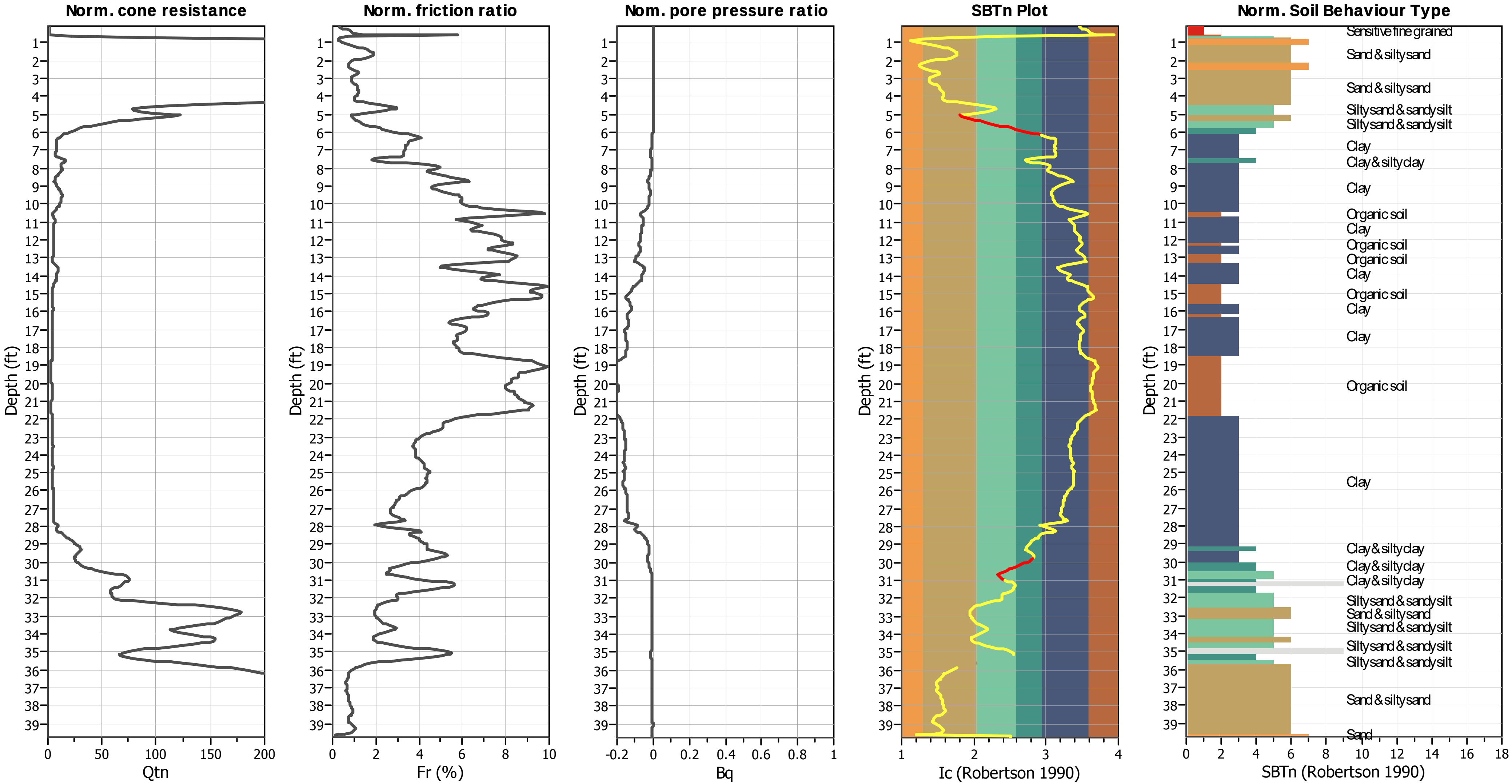
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



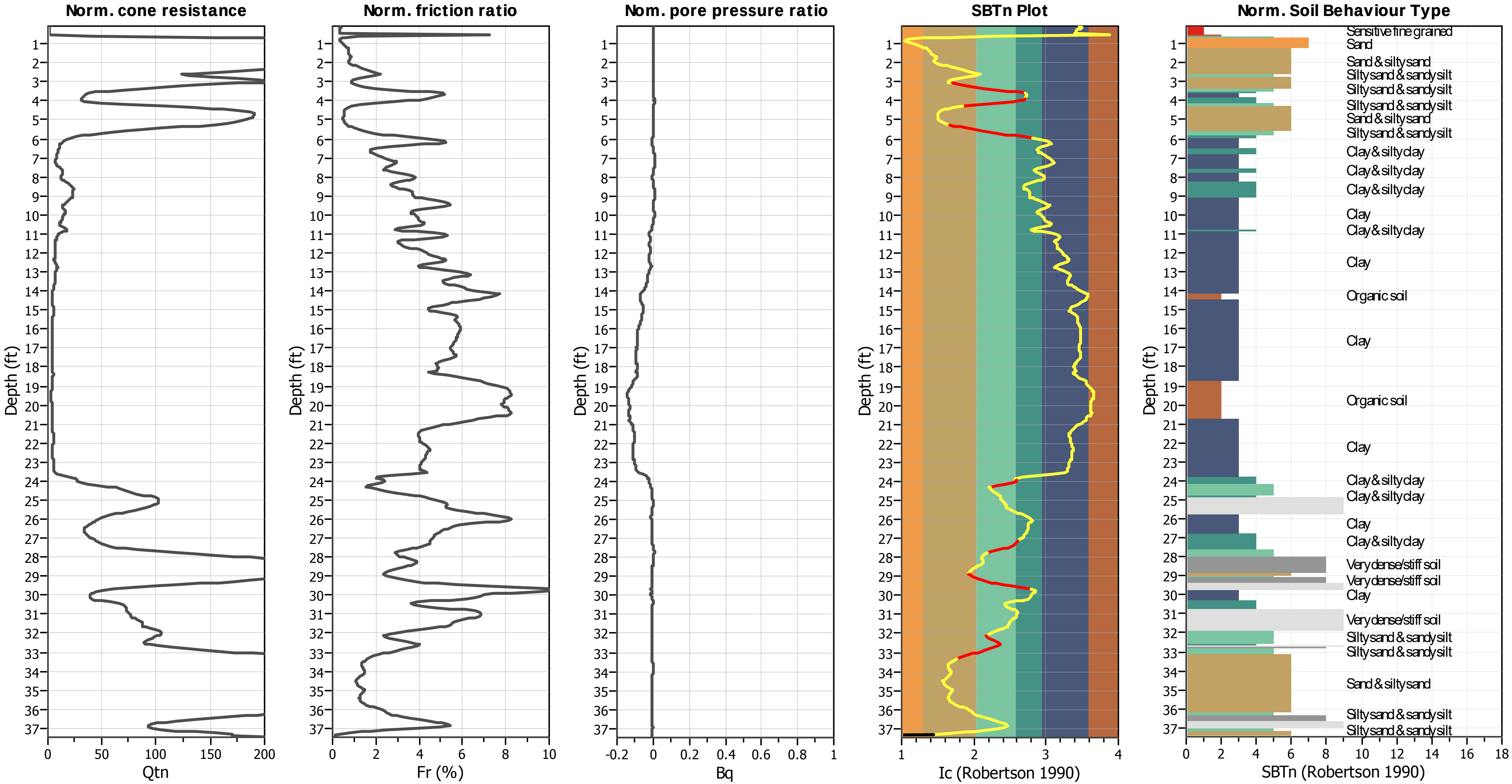
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



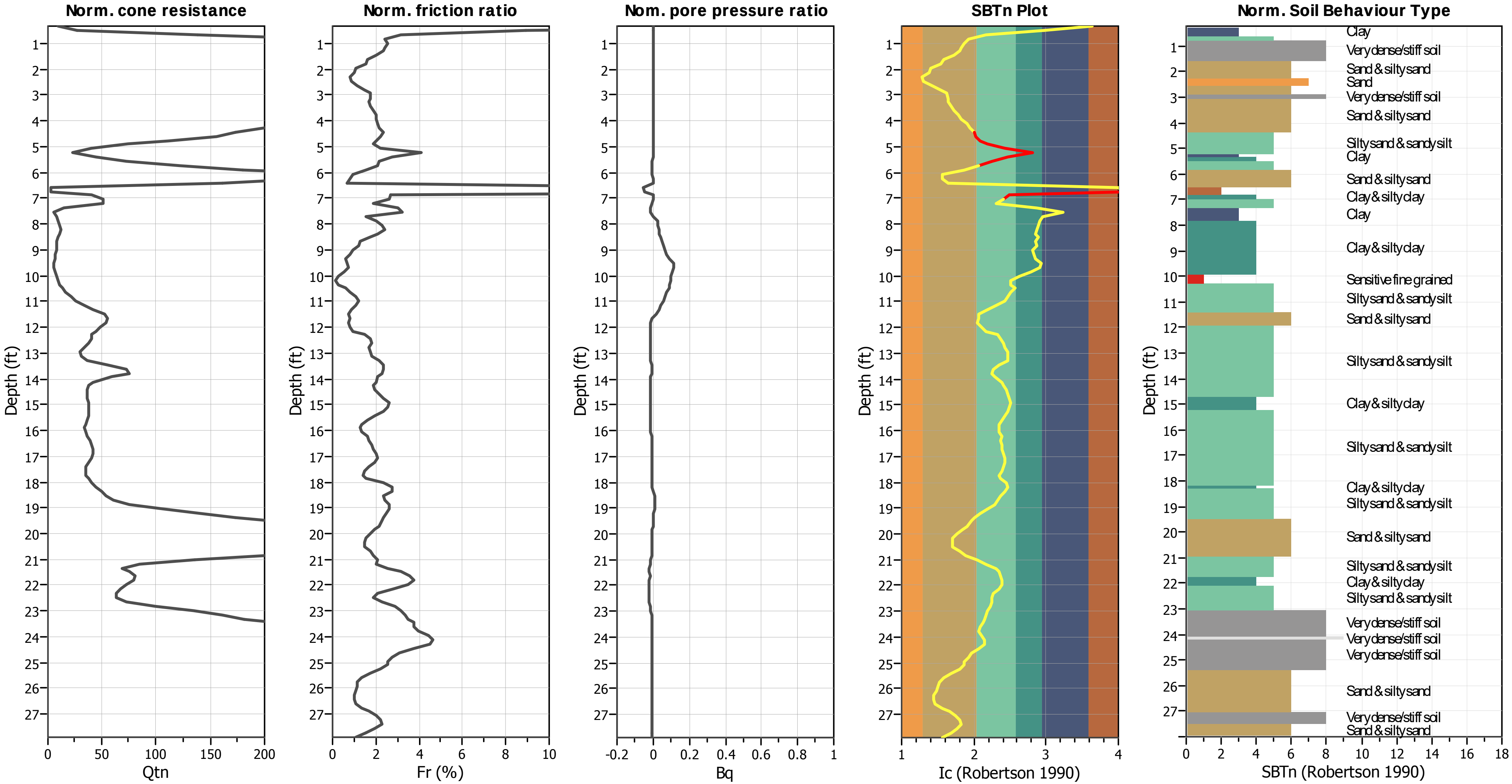
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



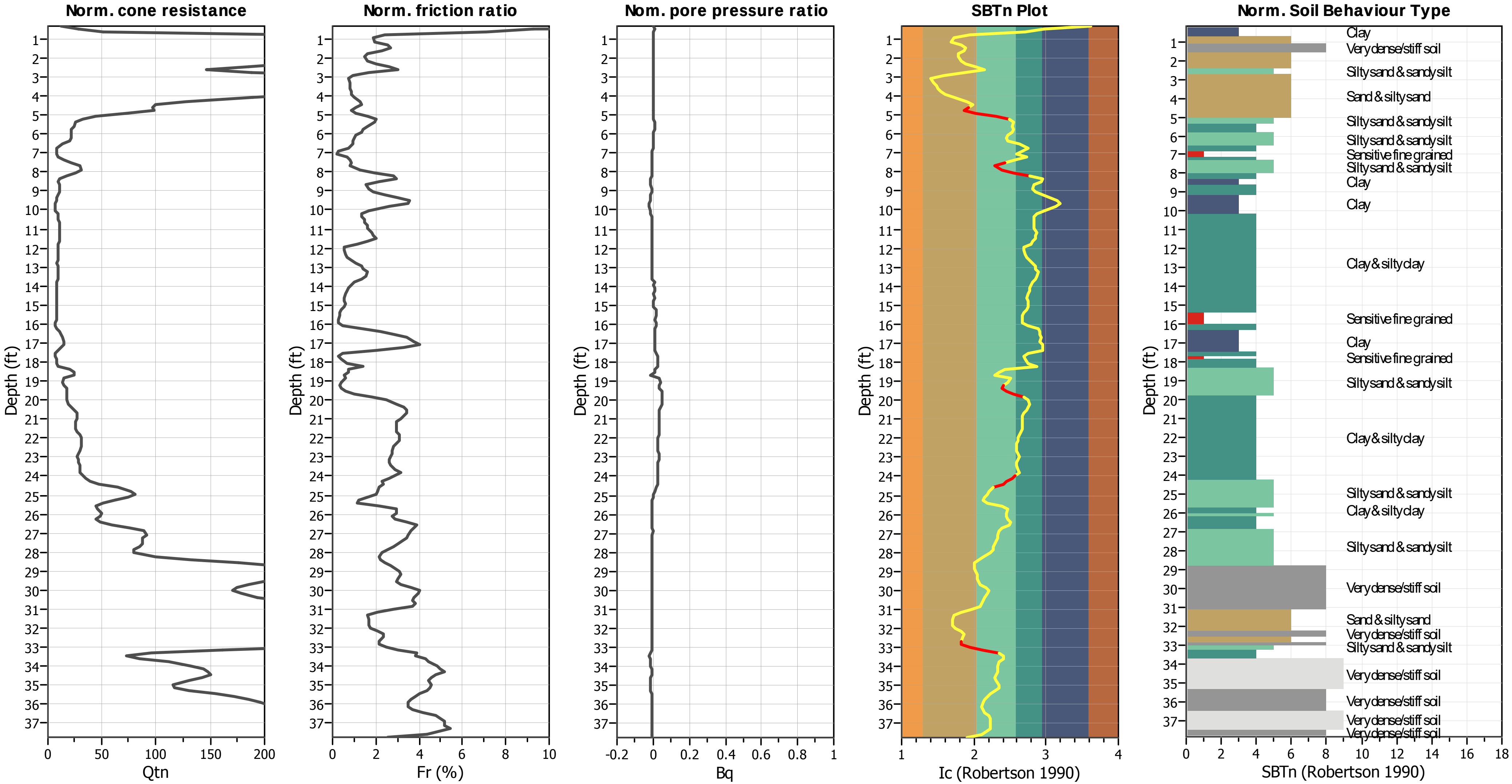
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



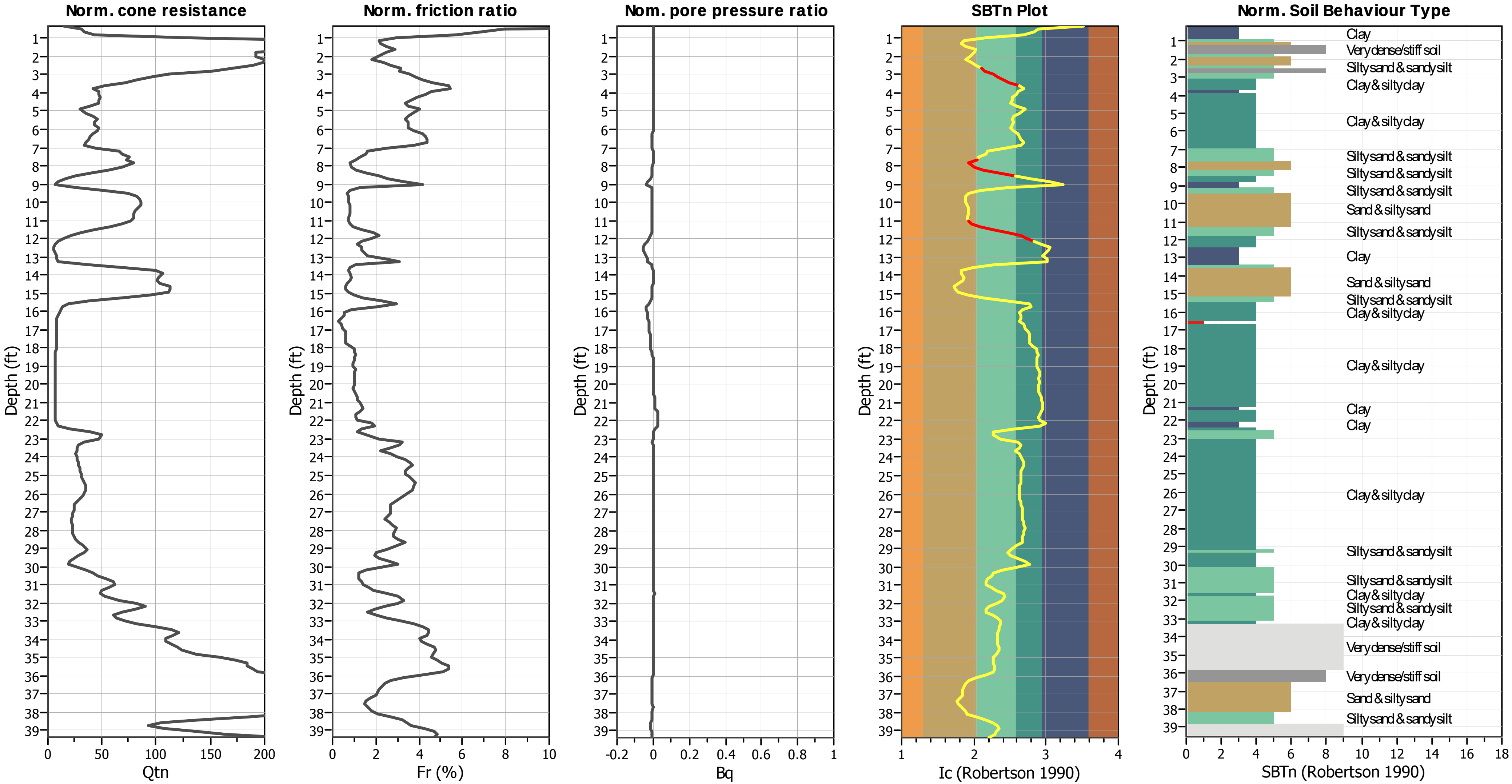
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



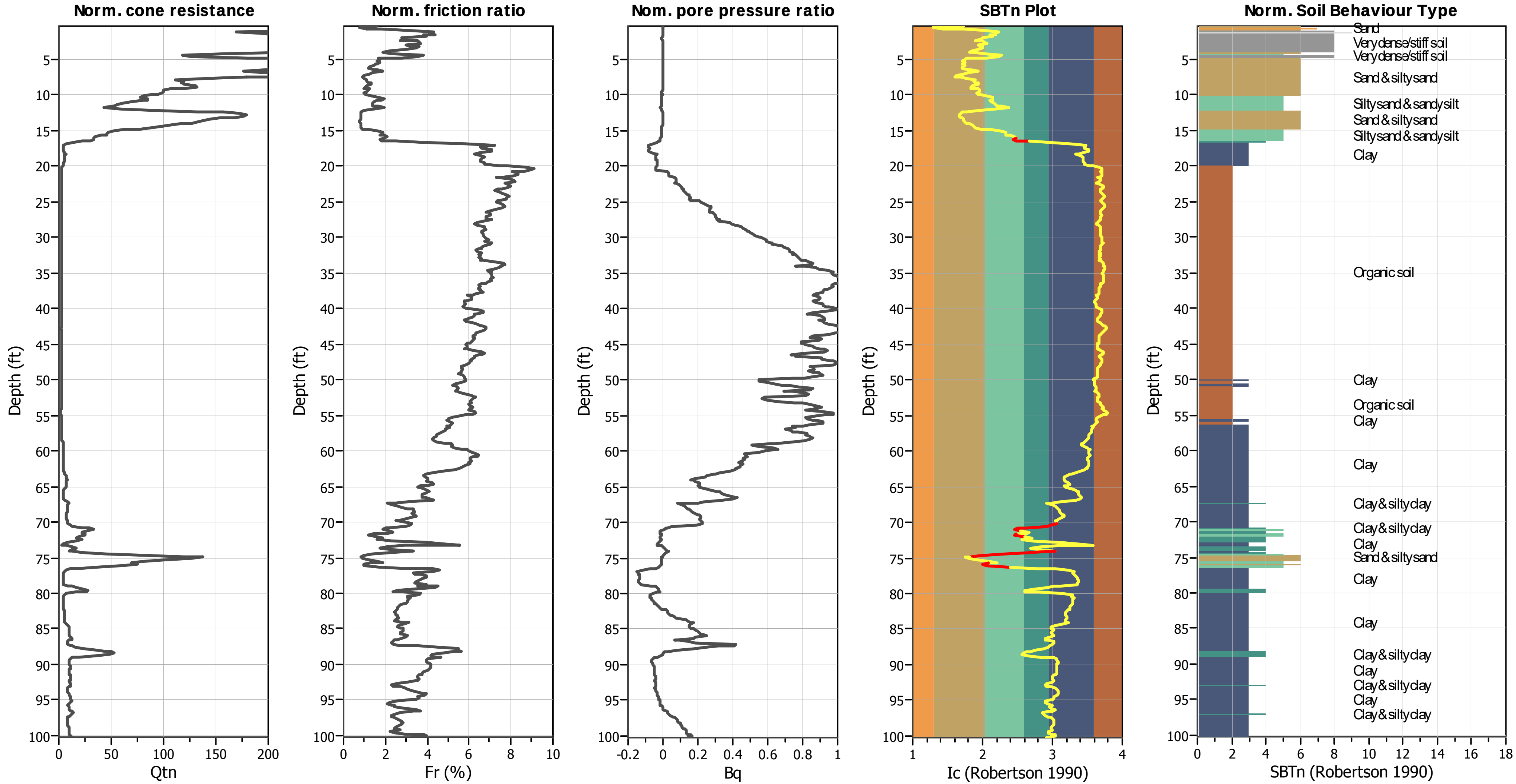
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



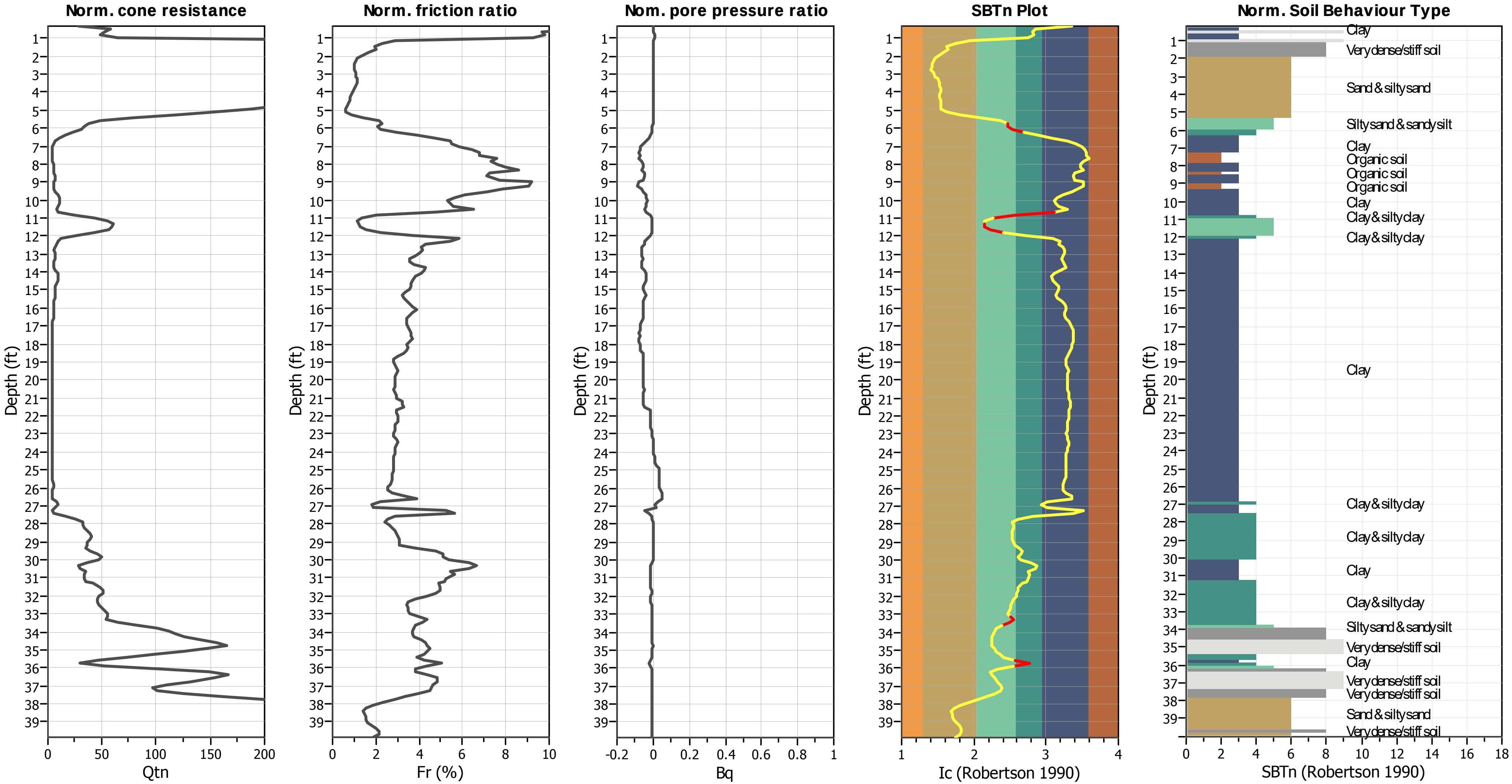
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.56	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	25.50 ft

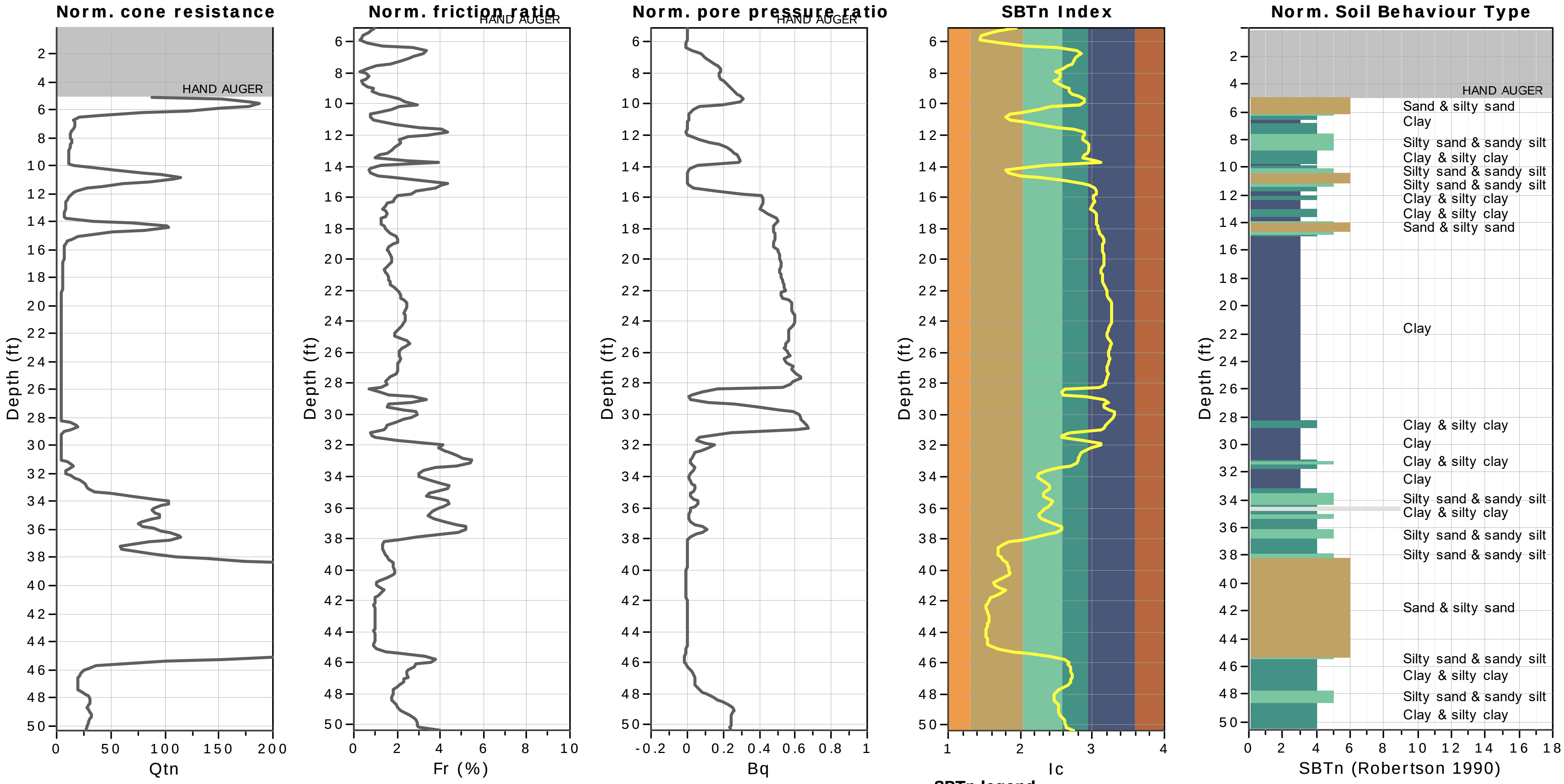
SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583

Project:
Location:

CPT: CPT0401
Total depth: 50.36 ft, Date: 10/11/2013
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



SBTn legend

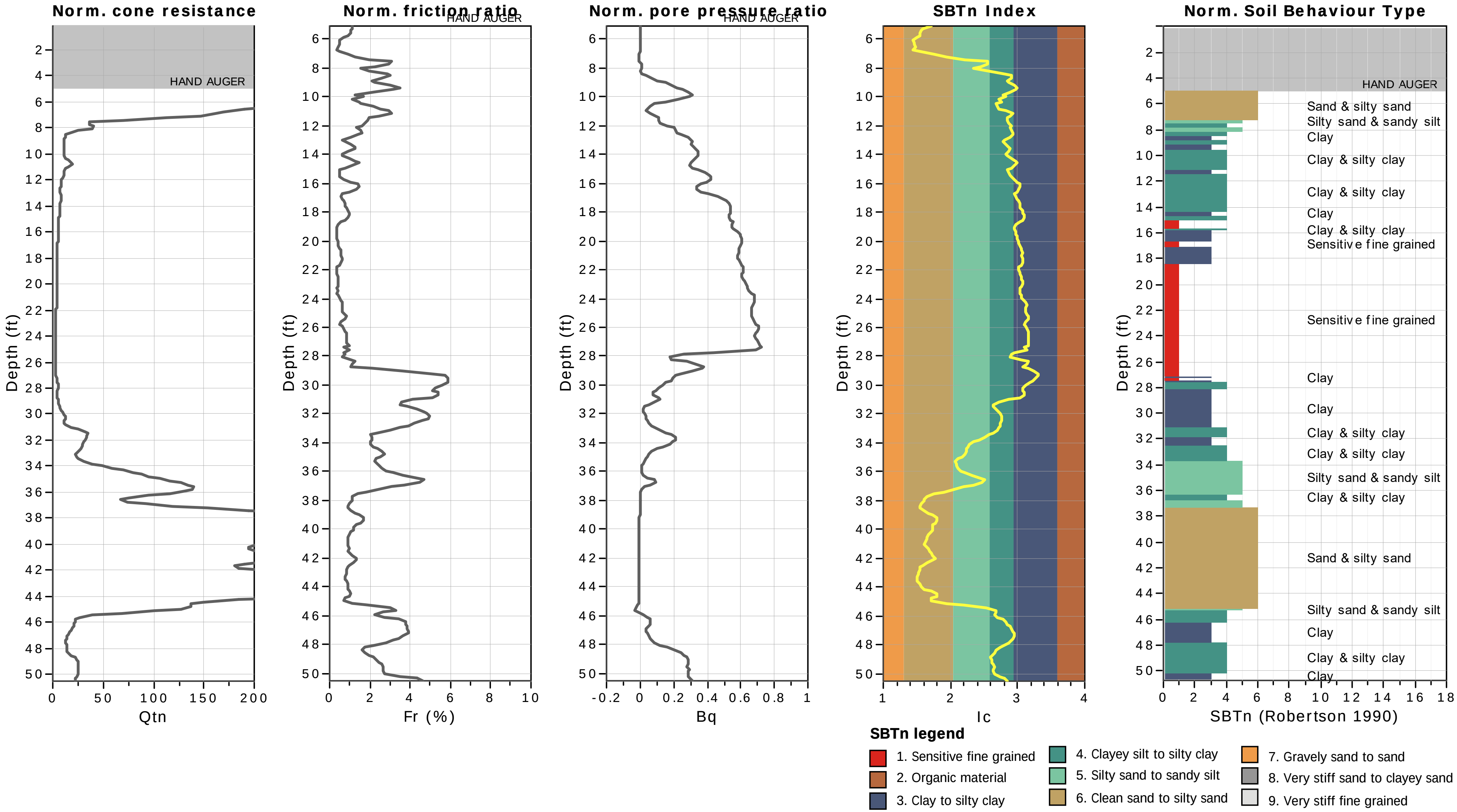
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583

Project:

Location:

CPT: CPT0402
Total depth: 50.52 ft, Date: 10/11/2013
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



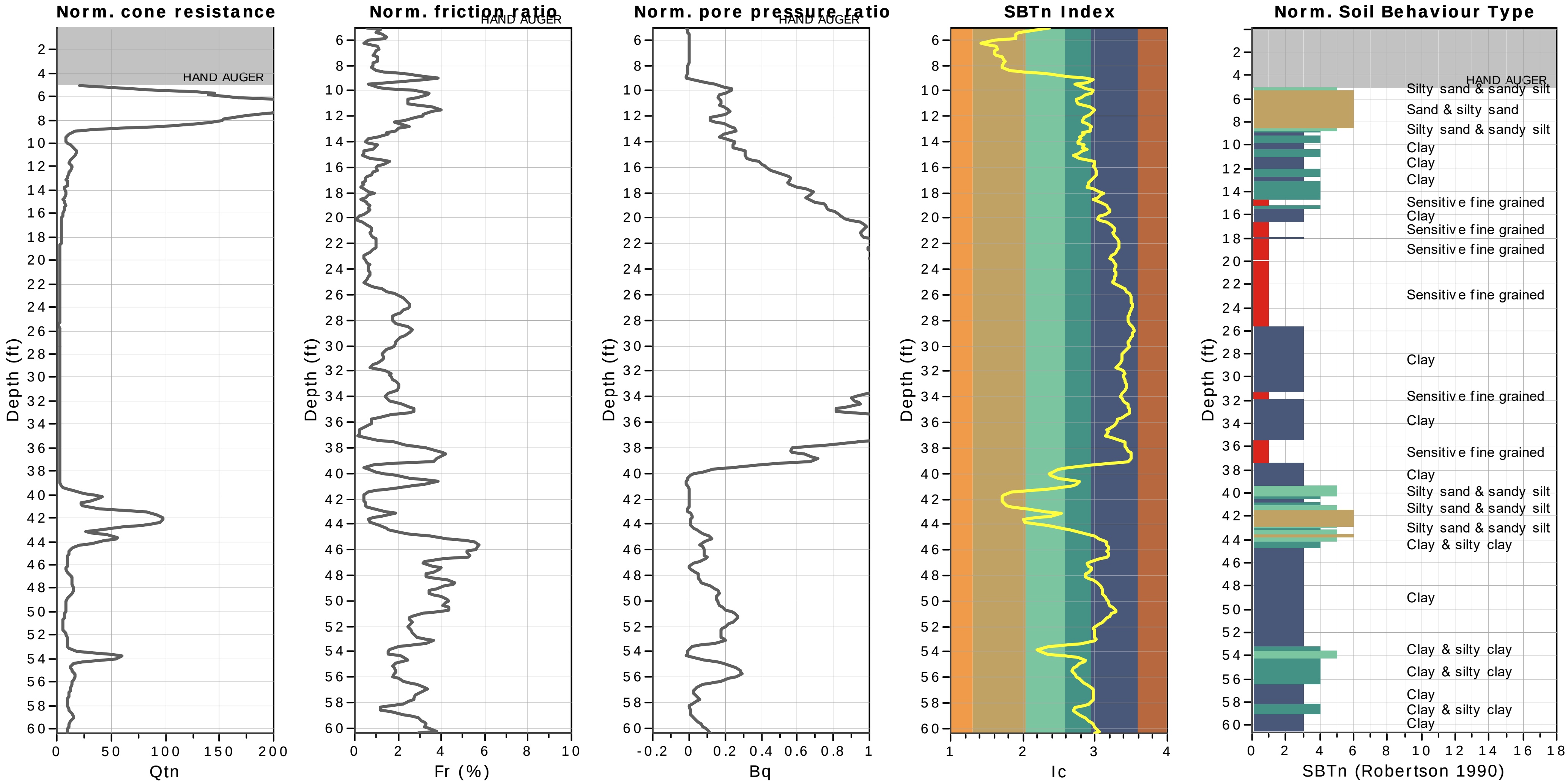
ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583

Project:

Location:

CPT: CPT0403

Total depth: 60.37 ft, Date: 10/11/2013
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



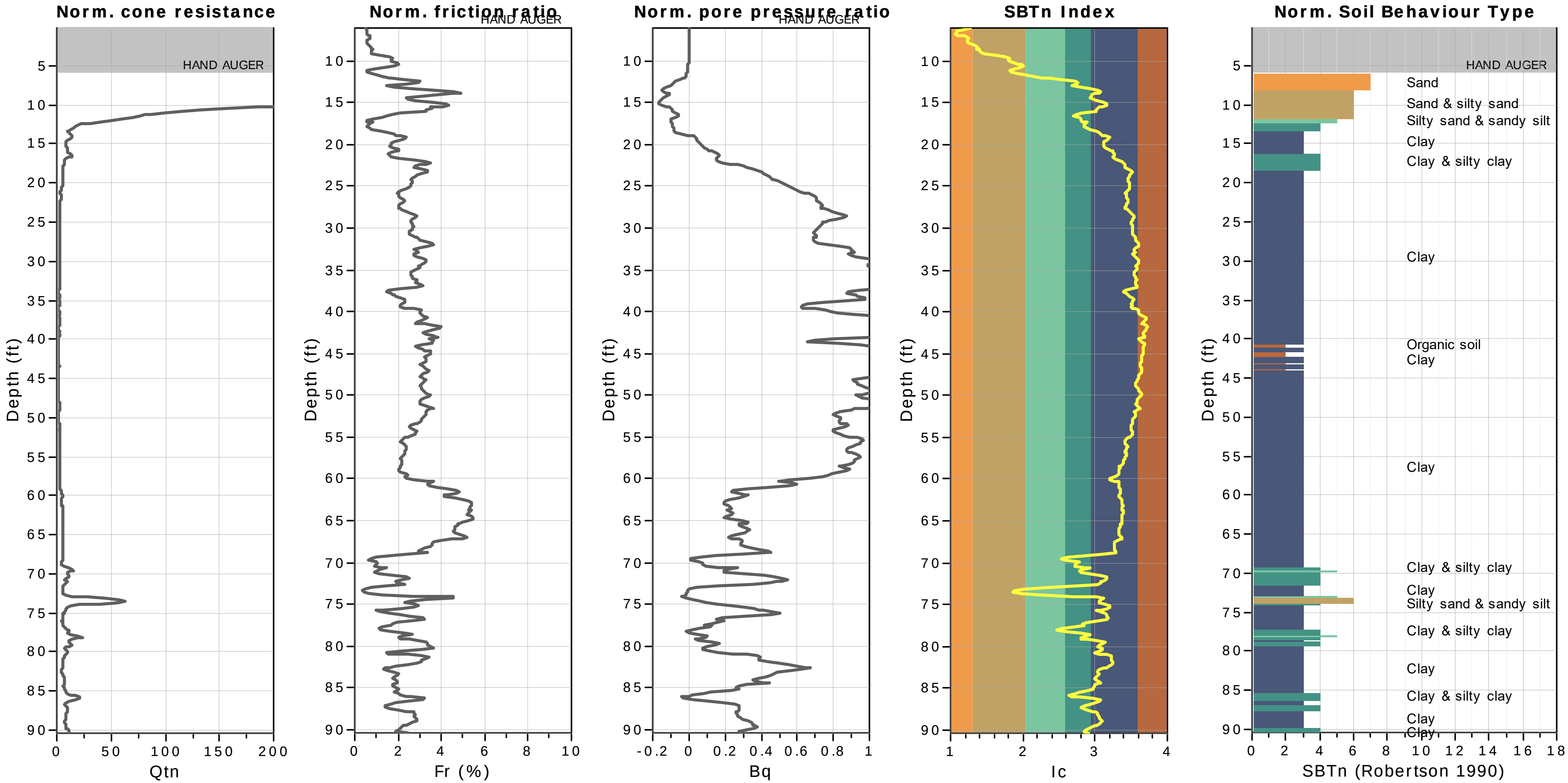
SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583

Project:
Location:

CPT: CPT0404
Total depth: 90.39 ft, Date: 10/11/2013
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



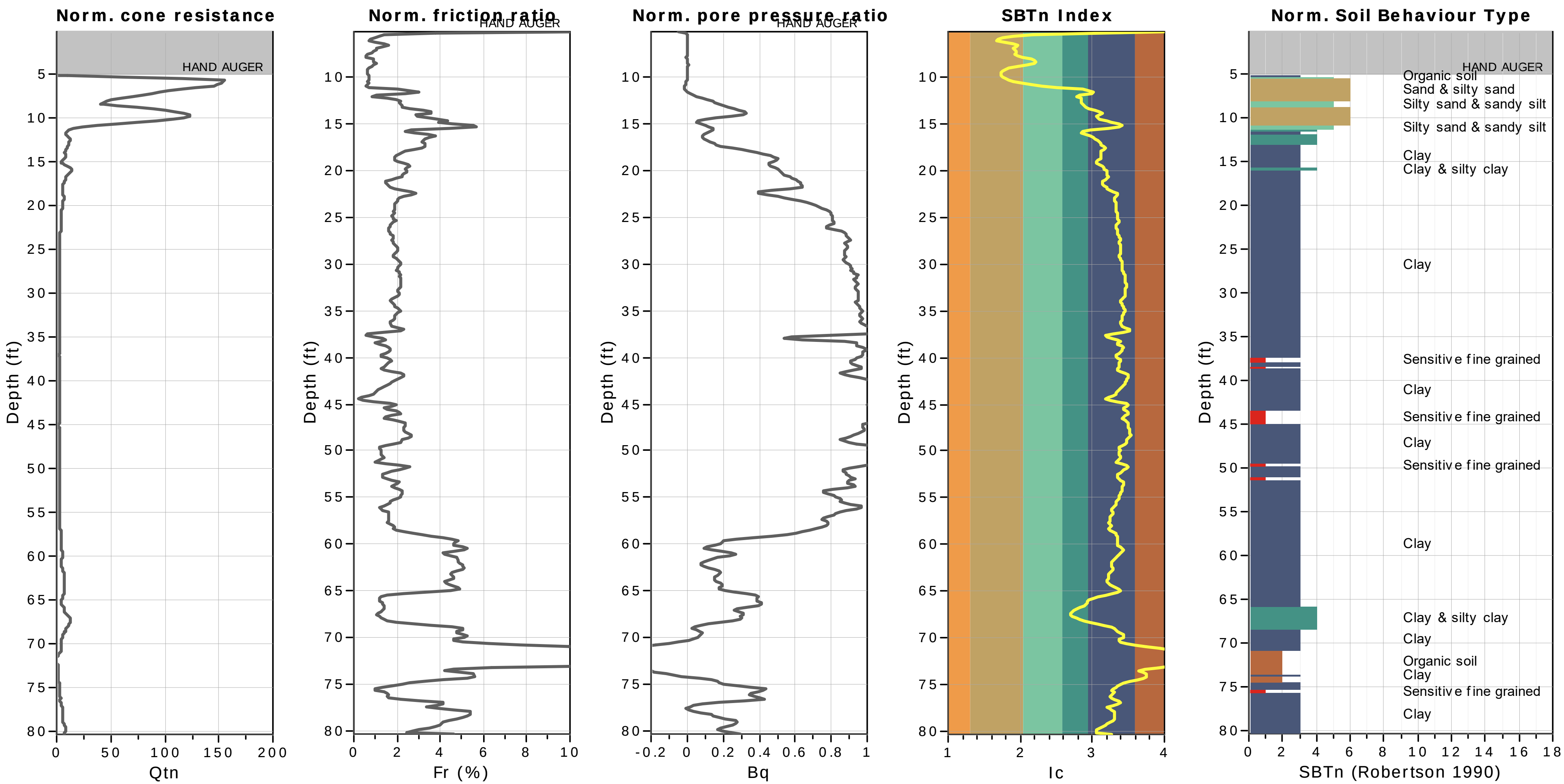
SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583

Project:
Location:

CPT: CPT0405
Total depth: 80.38 ft, Date: 10/11/2013
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



SBTn legend

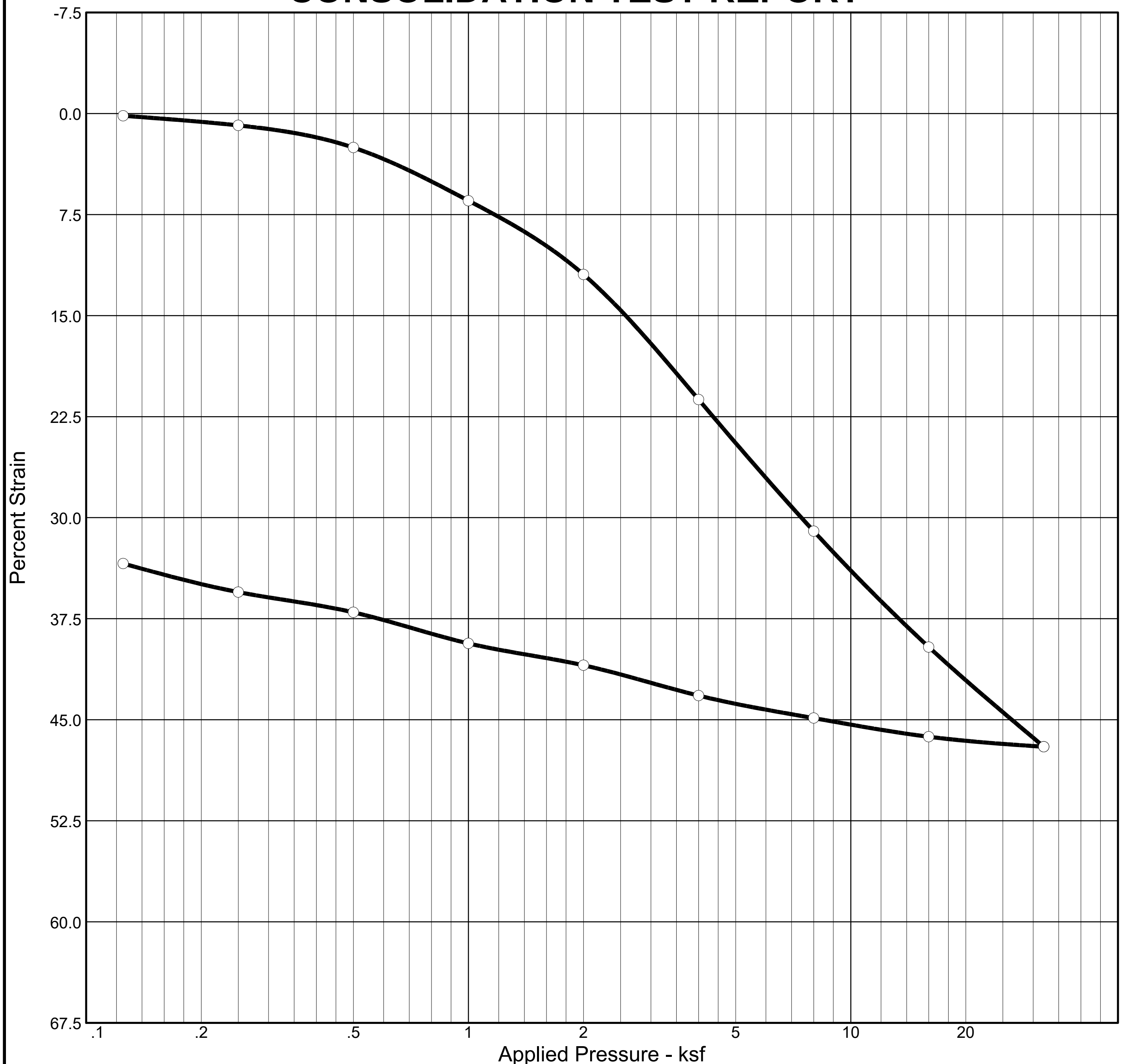
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

APPENDIX B

LABORATORY TEST DATA

APPENDIX B

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
97.4 %	115.2 %	39.6			2.525			2.984

MATERIAL DESCRIPTION

See exploration logs.

Project No. 9769.000.000 **Client:** STL Company, LLC

Project: Encinal Terminals

Source: B1-4 **Sample No.:** B1-4 @ 36.5

Remarks:

ASTM D2435, Method A; Initial dial reading = 0.02000; Initial sample height = 0.7696

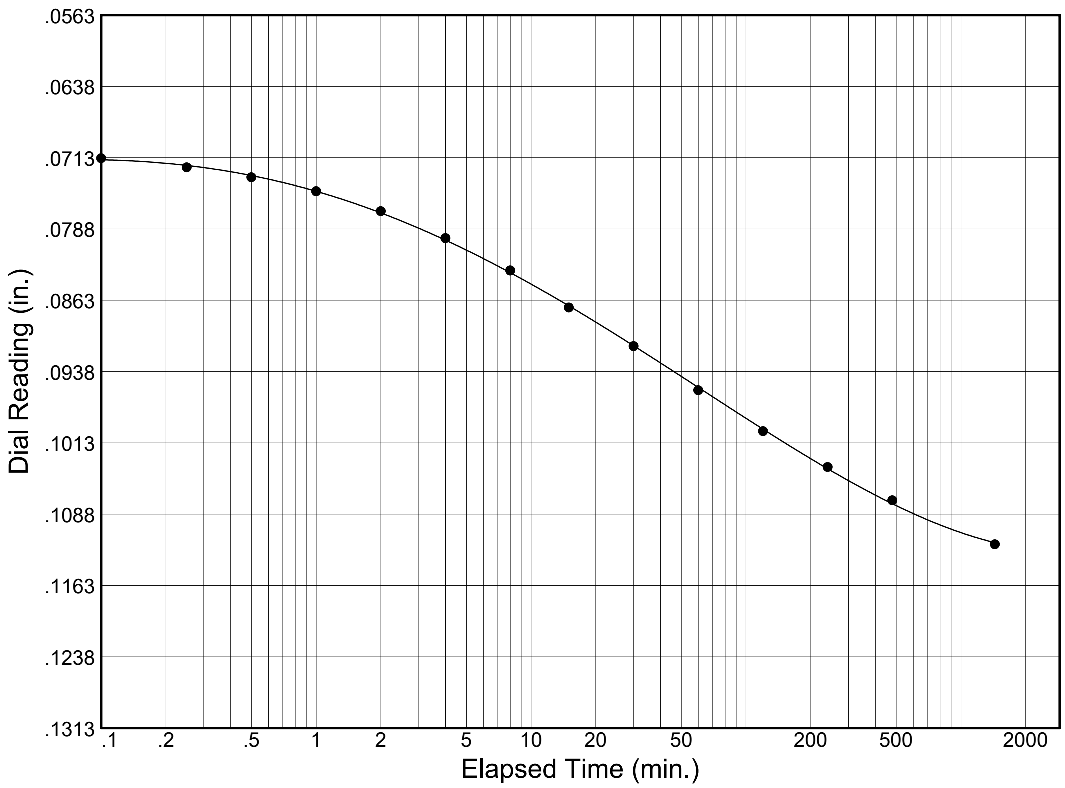
ENGEO INCORPORATED

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-4 Sample No.: B1-4 @ 36.5



Load No.= 5

Load= 2.00 ksf

D_0 = 0.06979

D_{50} = 0.08721

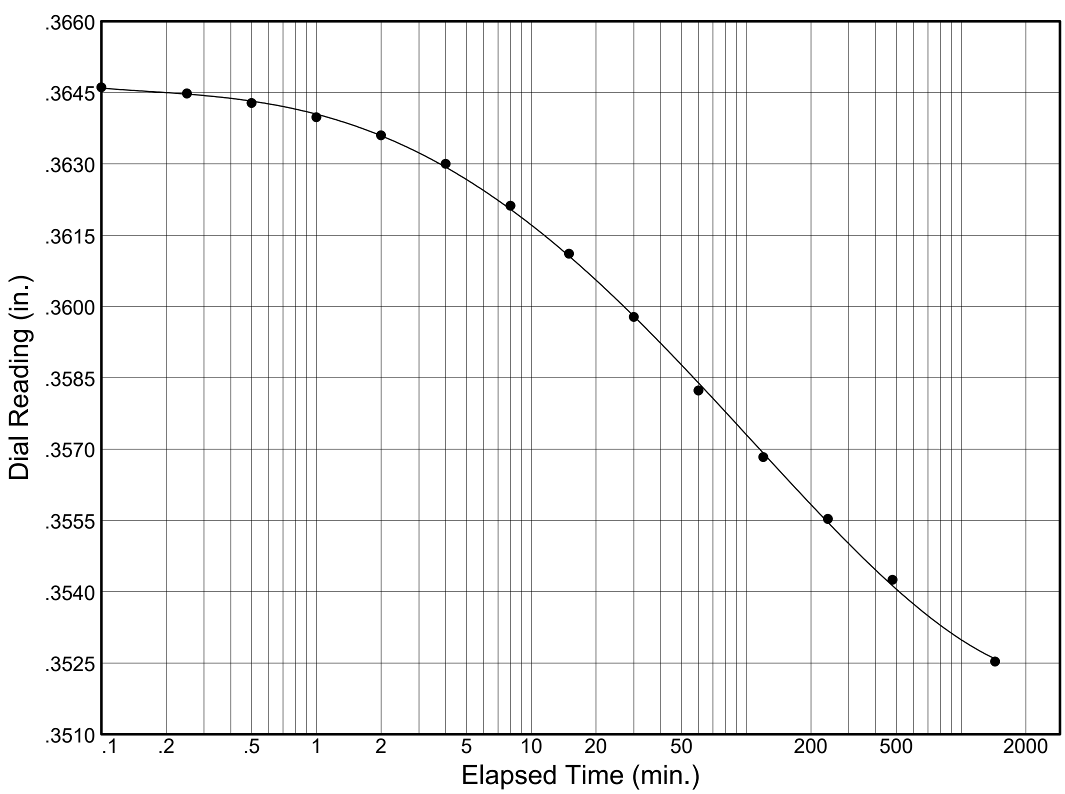
D_{100} = 0.10463

T_{50} = 15.83 min.

C_v @ T_{50}

0.02 ft.^{2}/day

C_α = 0.013



Load No.= 12

Load= 4.00 ksf

D_0 = 0.36538

D_{50} = 0.36023

D_{100} = 0.35507

T_{50} = 23.92 min.

C_v @ T_{50}

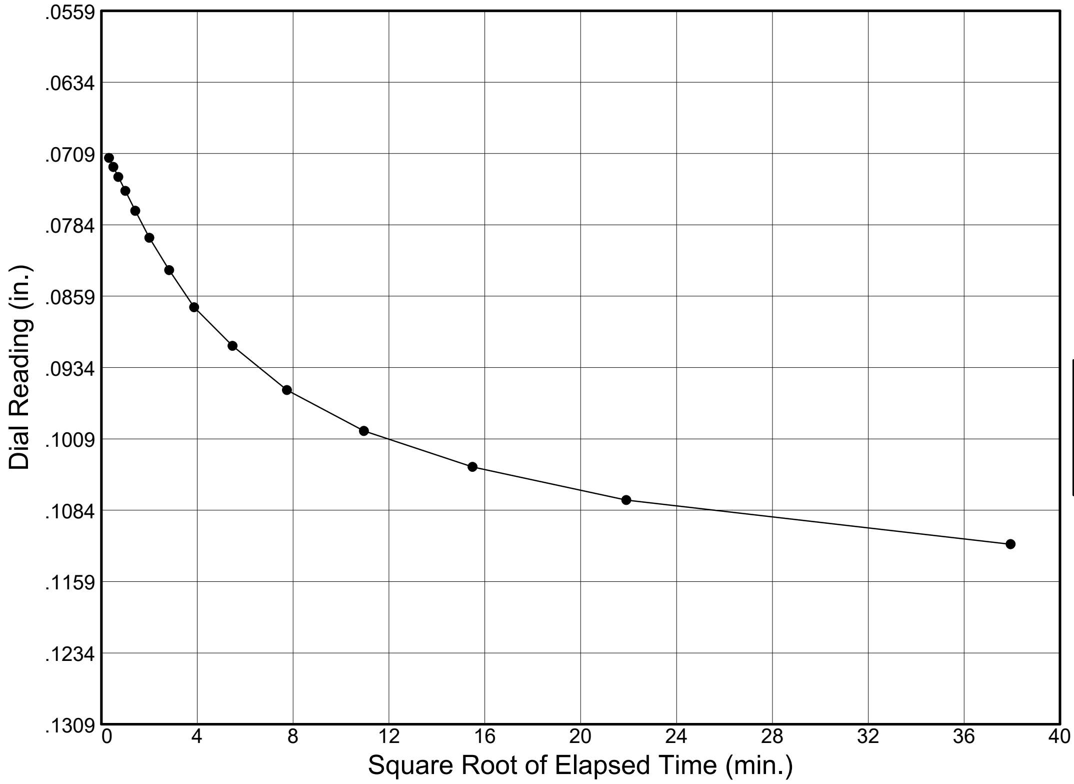
0.00 ft.^{2}/day

ENGEO, INC.

Dial Reading vs. Time

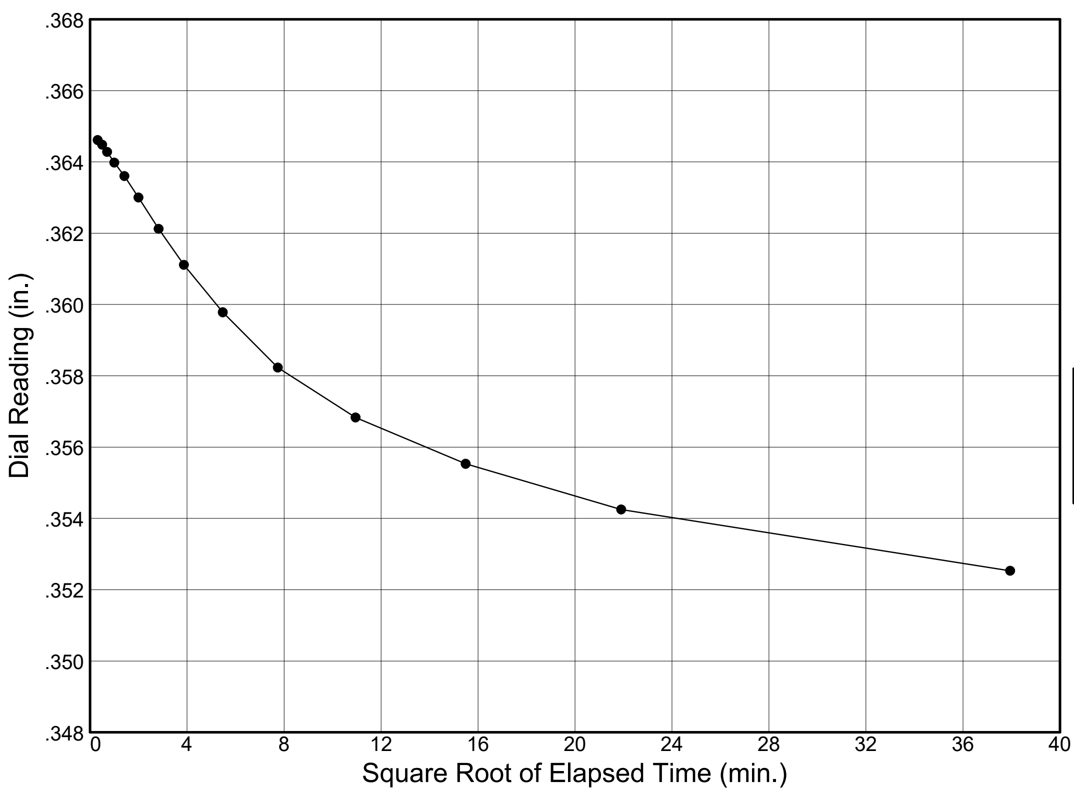
Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-4 Sample No.: B1-4 @ 36.5



Load No.= 5
Load= 2.00 ksf
D_0 = 0.07094
D_{90} = 0.09317
D_{100} = 0.09564
T_{90} = 41.94 min.

C_v @ T_{90}
0.02 ft.^{2}/day

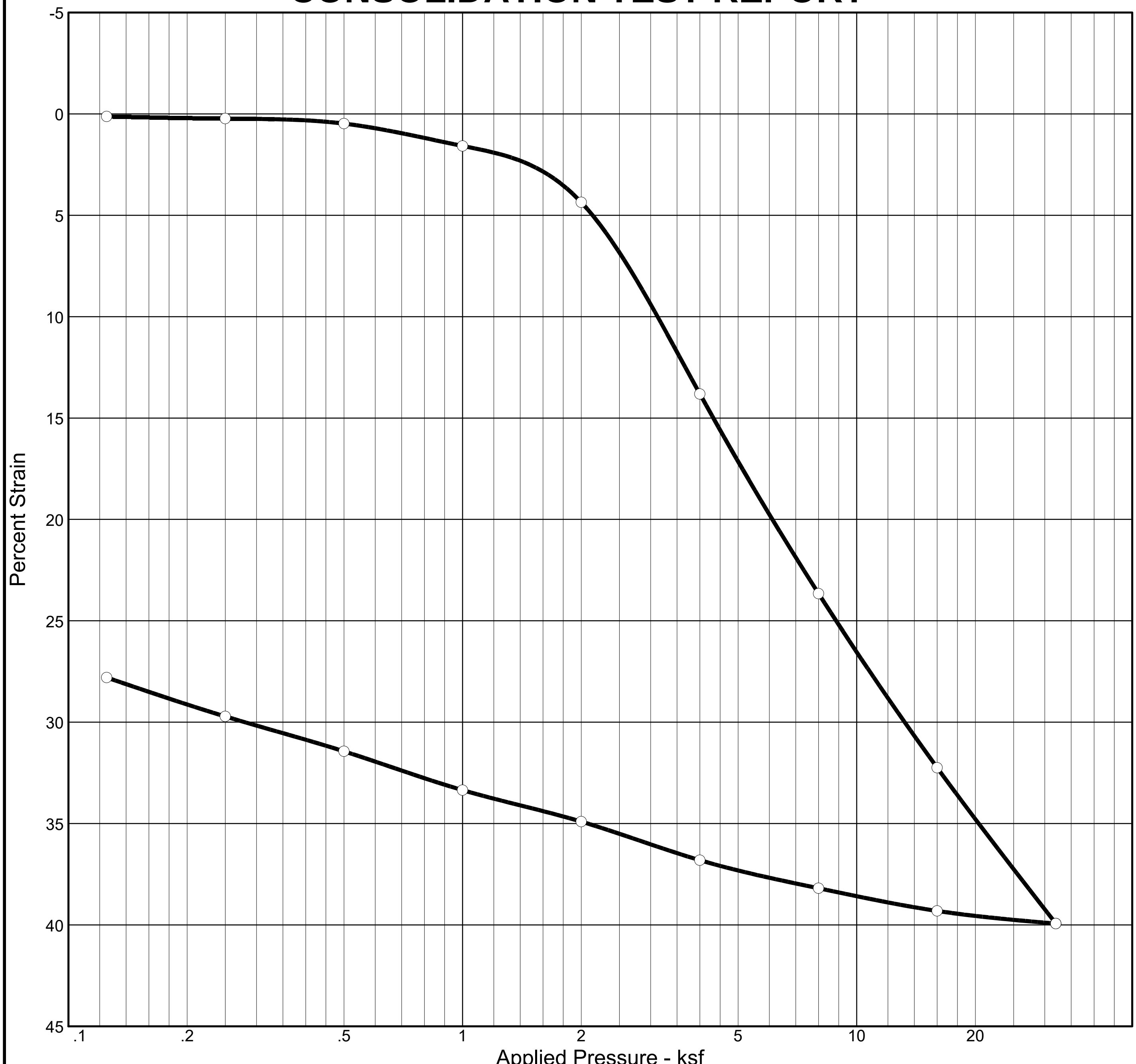


Load No.= 12
Load= 4.00 ksf
D_0 = 0.36482
D_{90} = 0.35745
D_{100} = 0.35663
T_{90} = 90.81 min.

C_v @ T_{90}
0.00 ft.^{2}/day

ENGEO, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
98.6 %	96.2 %	46.2			2.662			2.597

MATERIAL DESCRIPTION

See exploration logs.

Project No. 9769.000.000 **Client:** STL Company, LLC

Project: Encinal Terminals

Source: B1-5 **Sample No.:** B1-5 @ 26.5

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

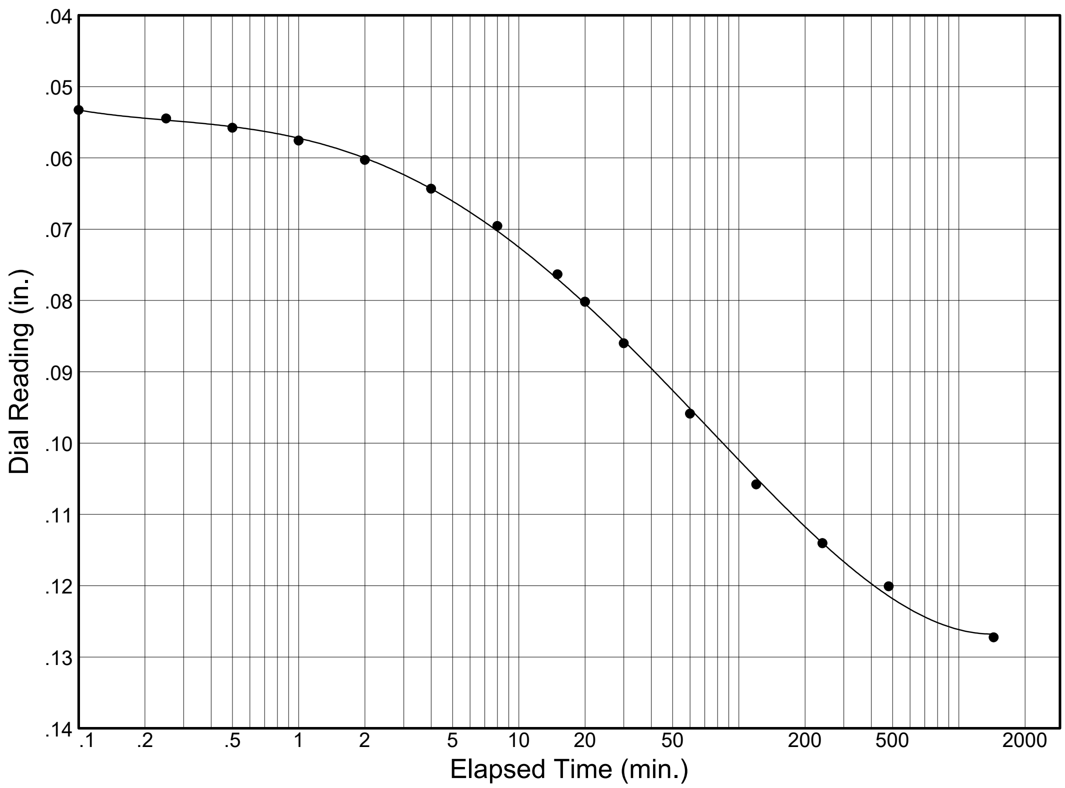
Remarks:

ASTM D2435, Method A; Initial dial reading = 0.02000; Initial sample height = 0.7764

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-5 Sample No.: B1-5 @ 26.5



Load No.= 6

Load= 4.00 ksf

D_0 = 0.04985

D_{50} = 0.08307

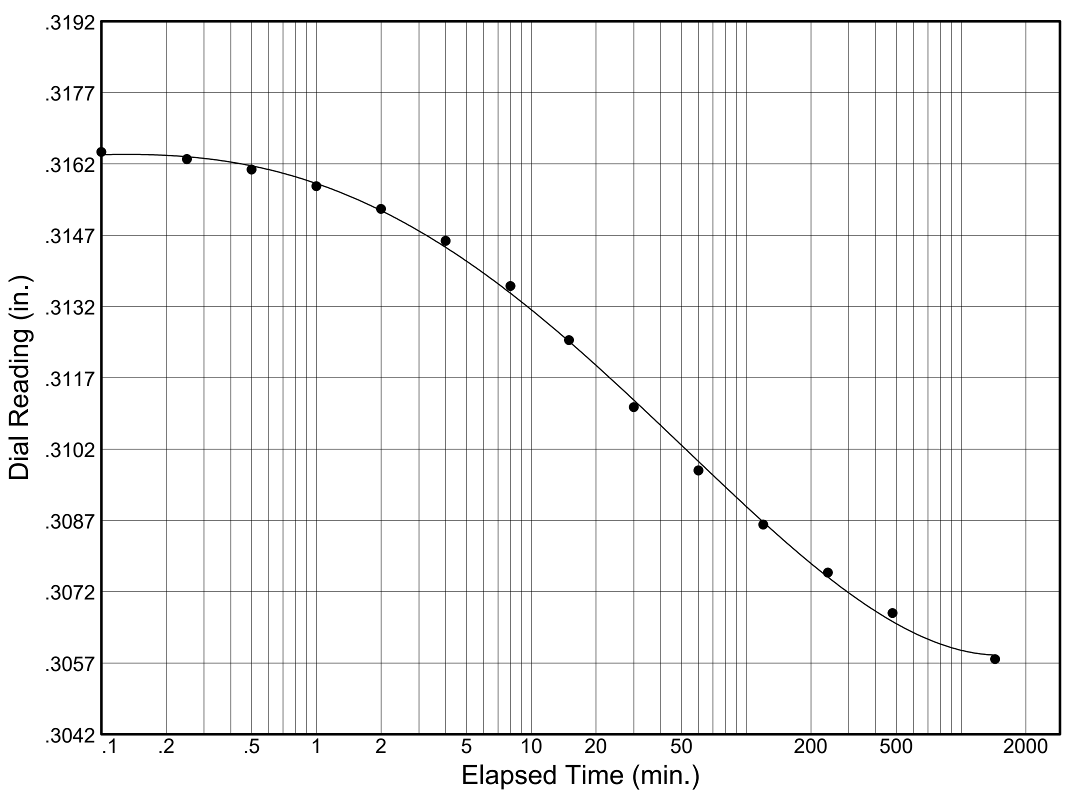
D_{100} = 0.11628

T_{50} = 24.63 min.

C_v @ T_{50}

0.01 ft.^{2}/day

C_α = 0.020



Load No.= 12

Load= 4.00 ksf

D_0 = 0.31652

D_{50} = 0.31195

D_{100} = 0.30738

T_{50} = 20.16 min.

C_v @ T_{50}

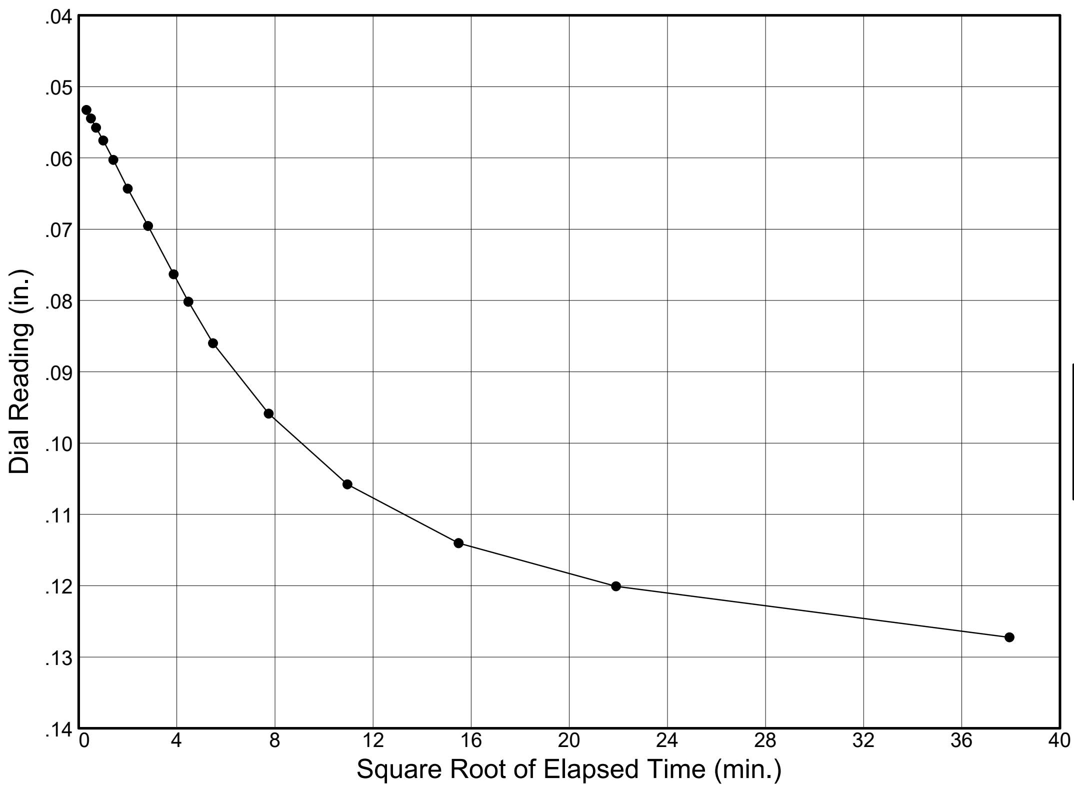
0.01 ft.^{2}/day

ENGEO, INC.

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-5 Sample No.: B1-5 @ 26.5



Load No.= 6

Load= 4.00 ksf

D_0 = 0.05206

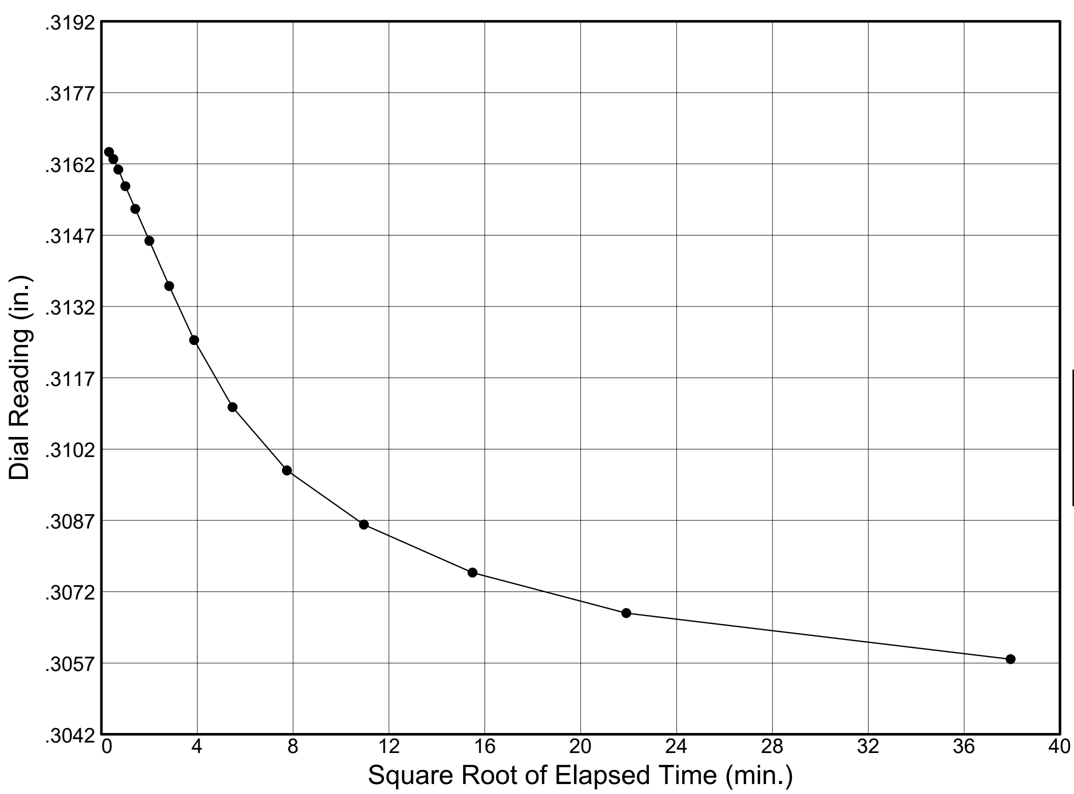
D_{90} = 0.10122

D_{100} = 0.10668

T_{90} = 89.76 min.

C_v @ T_{90}

0.01 ft.^{2}/day



Load No.= 12

Load= 4.00 ksf

D_0 = 0.31677

D_{90} = 0.31006

D_{100} = 0.30931

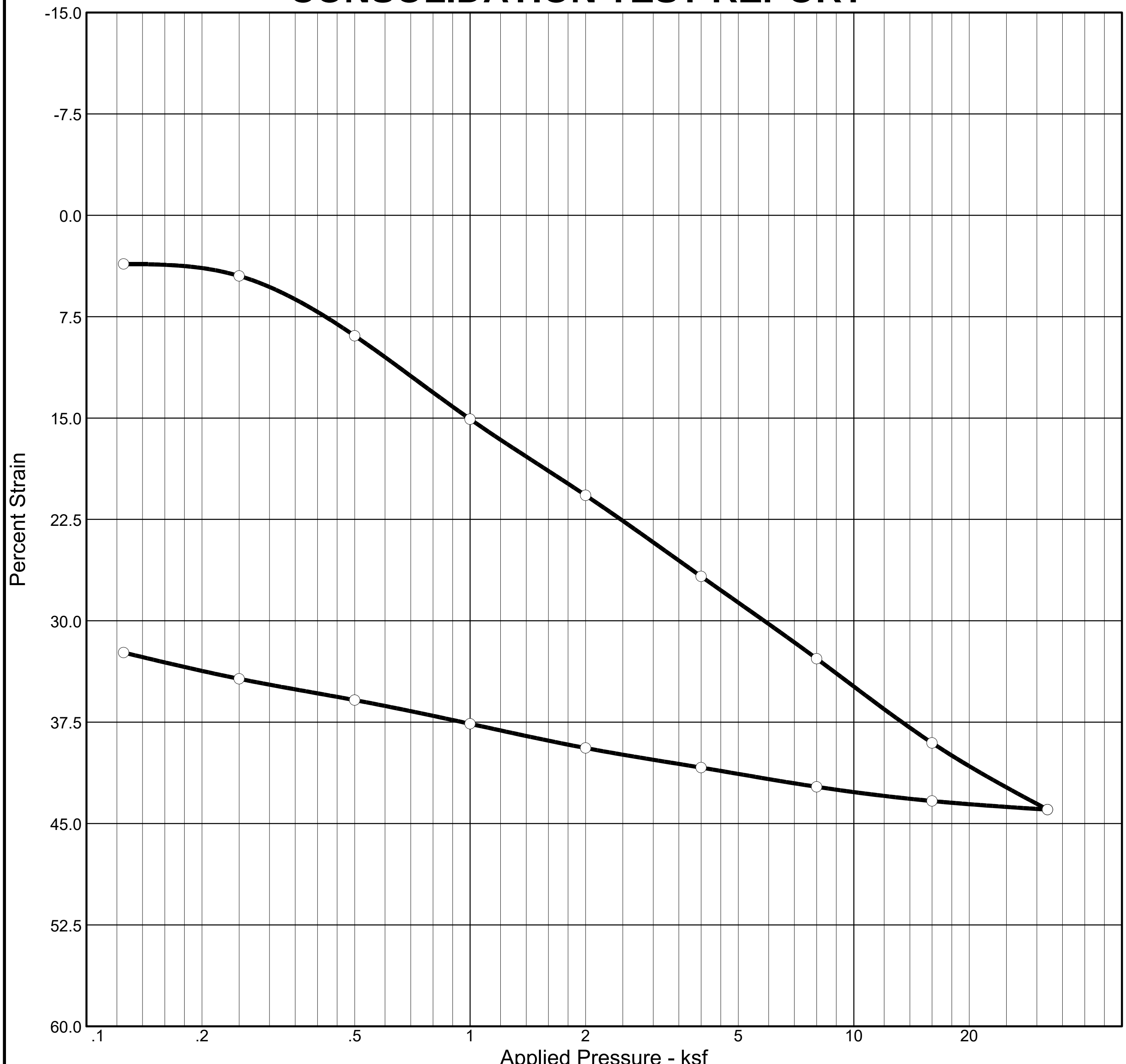
T_{90} = 52.11 min.

C_v @ T_{90}

0.01 ft.^{2}/day

ENGEO, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
97.2 %	94.2 %	46.5			2.669			2.585

MATERIAL DESCRIPTION

See exploration logs.

Project No. 9769.000.000 **Client:** STL Company, LLC

Project: Encinal Terminals

Source: B1-5 **Sample No.:** B1-5 @ 56

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Remarks:

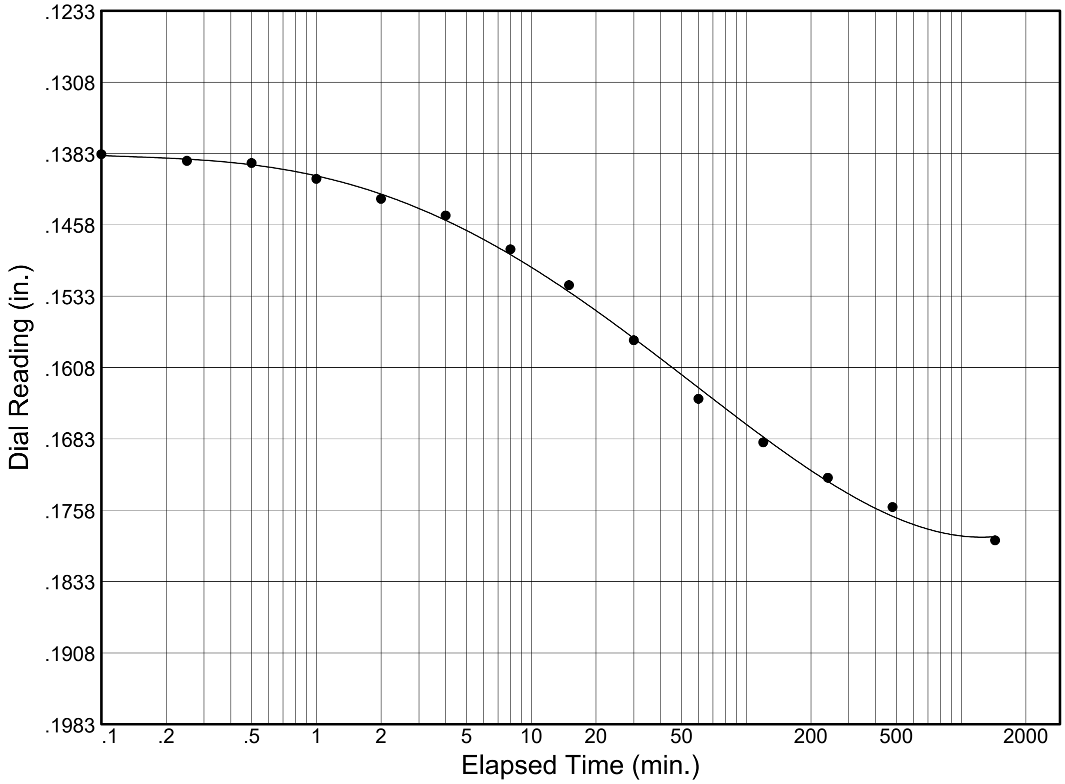
ASTM D2435, Method A; Very soft and disturbed; Initial dial reading = 0.02000; Initial sample height = 0.7673

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-5

Sample No.: B1-5 @ 56.5



Load No.= 5

Load= 2.00 ksf

D_0 = 0.13573

D_{50} = 0.15444

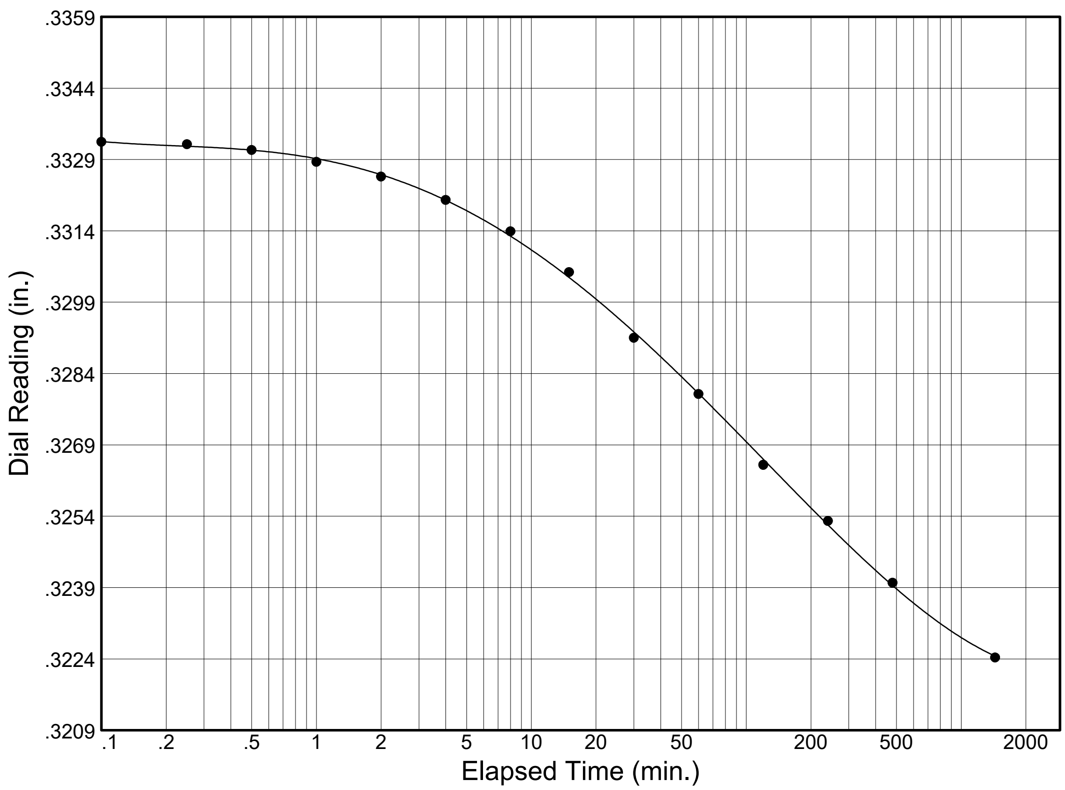
D_{100} = 0.17316

T_{50} = 18.96 min.

C_v @ T_{50}

0.01 ft.^{2}/day

C_α = 0.011



Load No.= 13

Load= 2.00 ksf

D_0 = 0.33354

D_{50} = 0.32901

D_{100} = 0.32448

T_{50} = 34.82 min.

C_v @ T_{50}

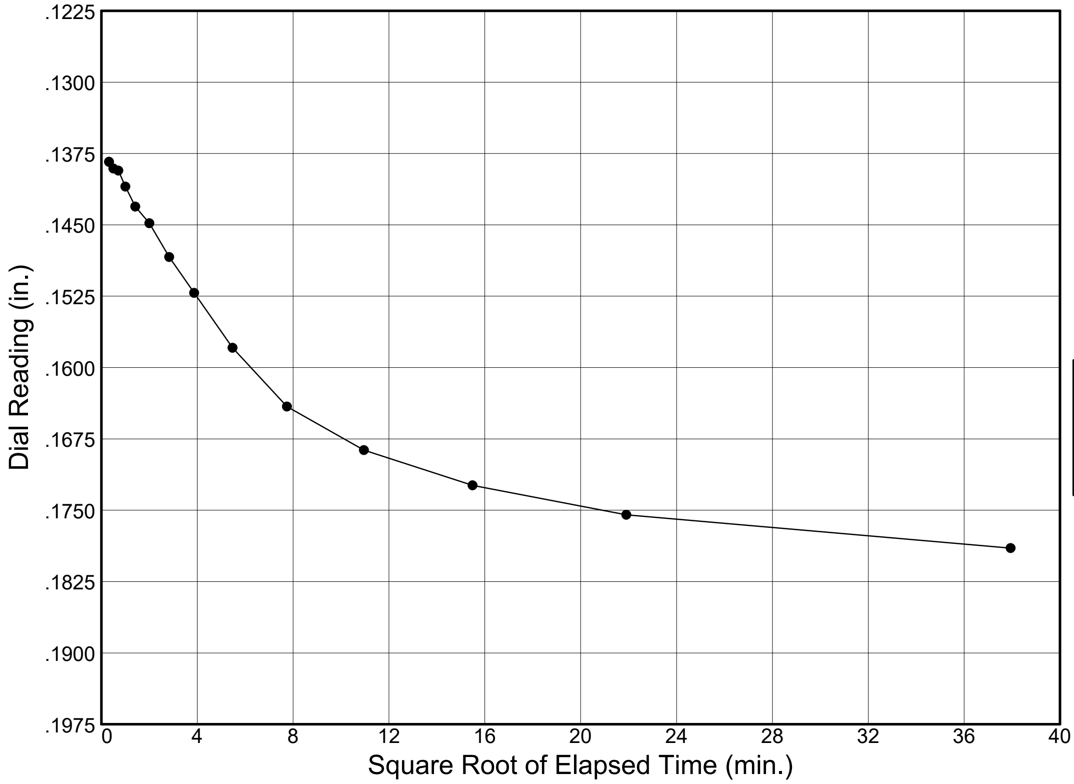
0.00 ft.^{2}/day

ENGEO, INC.

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-5 Sample No.: B1-5 @ 56.5



Load No.= 5

Load= 2.00 ksf

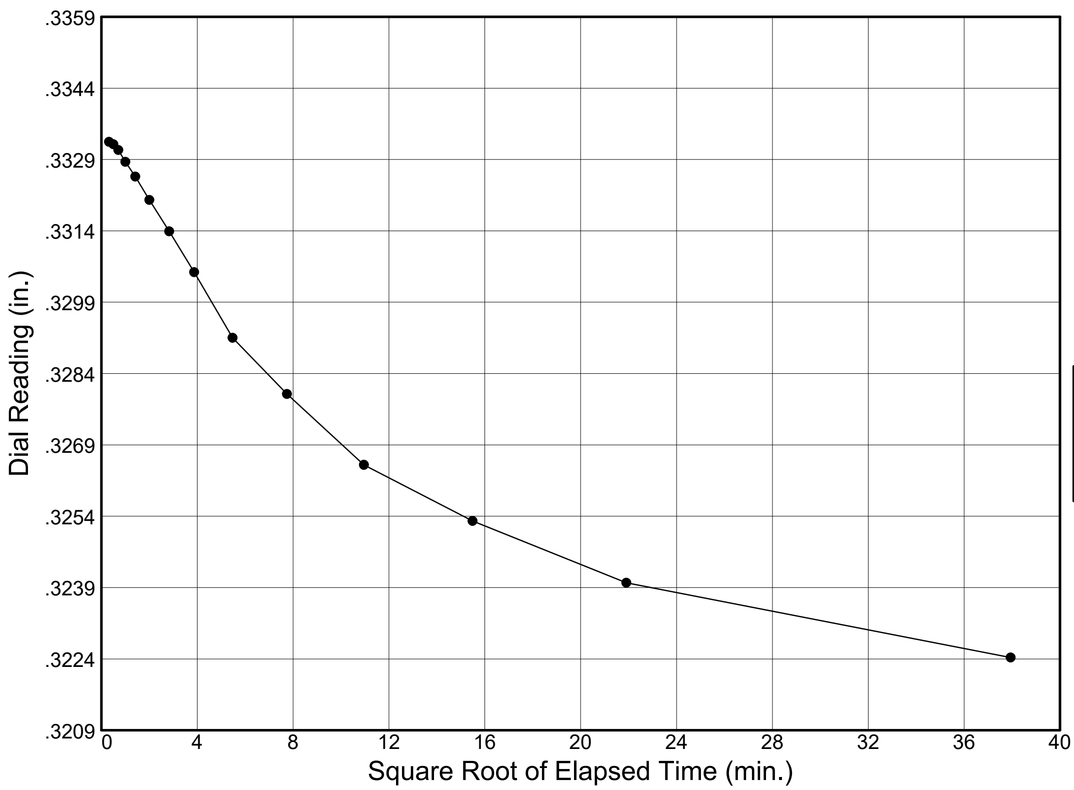
D_0 = 0.13758

D_{90} = 0.16619

D_{100} = 0.16937

T_{90} = 84.82 min.

C_v @ T_{90}
0.01 ft.^{2}/day



Load No.= 13

Load= 2.00 ksf

D_0 = 0.33355

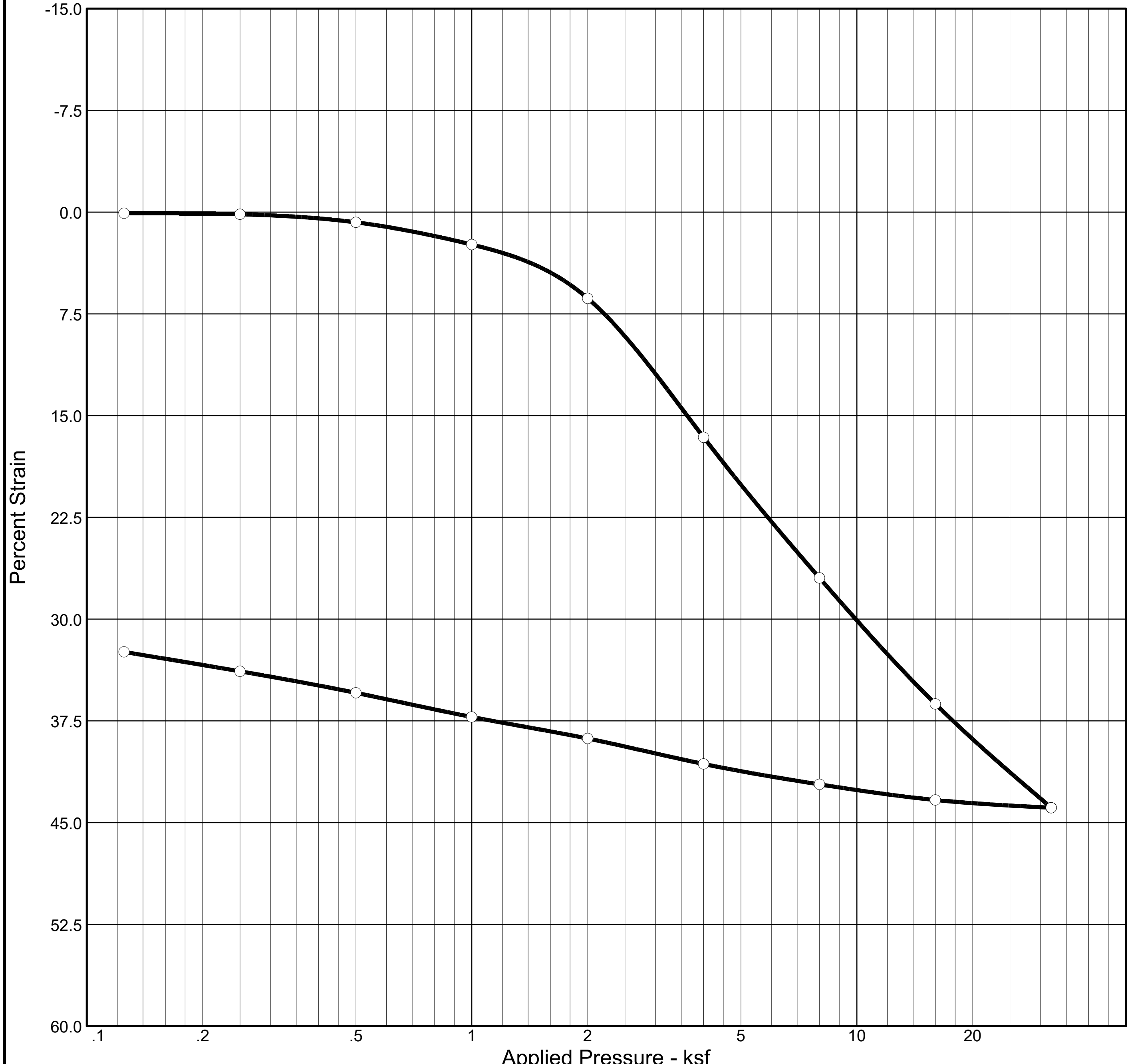
D_{90} = 0.32670

D_{100} = 0.32594

T_{90} = 109.71 min.

C_v @ T_{90}
0.00 ft.^{2}/day

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
99.6 %	110.4 %	41.9			2.624			2.908

MATERIAL DESCRIPTION

See exploration logs.

Project No. 9769.000.000 **Client:** STL Company, LLC

Project: Encinal Terminals

Source: B1-6 **Sample No.:** B1-6 @ 16

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

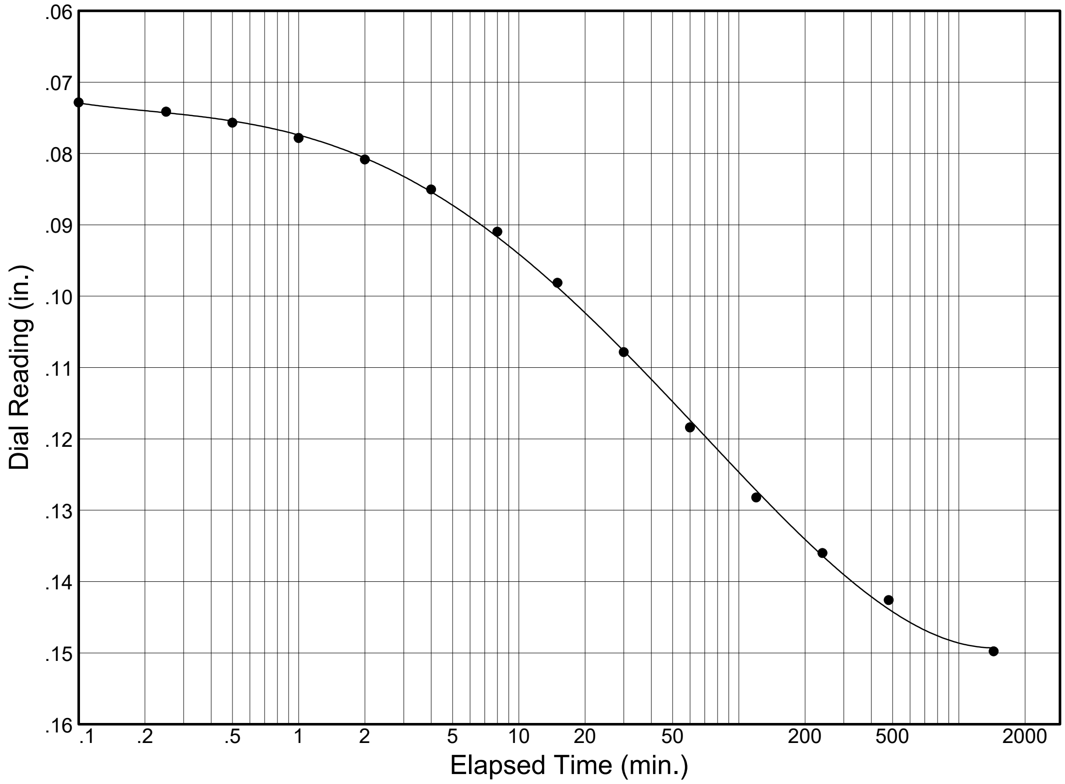
Remarks:

ASTM D2435, Method A; Initial dial reading = 0.02000; Initial sample height = 0.78175

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-6 Sample No.: B1-6 @ 16.5



Load No.= 6

Load= 4.00 ksf

D_0 = 0.06970

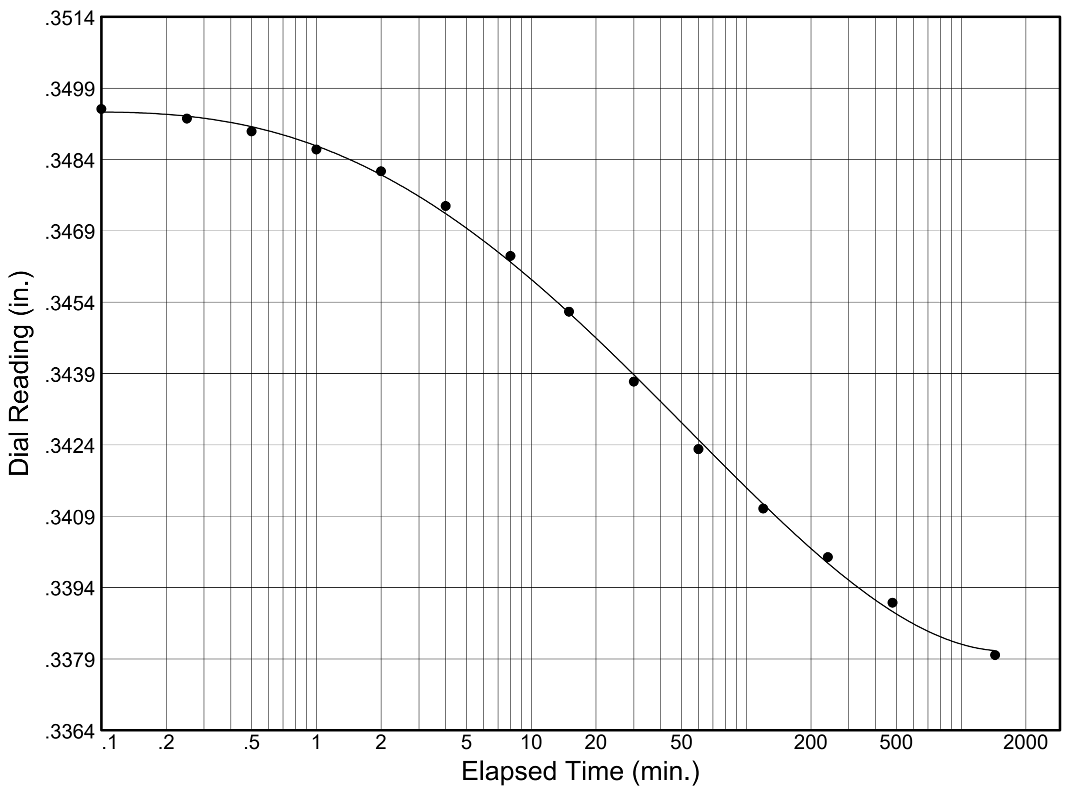
D_{50} = 0.10425

D_{100} = 0.13880

T_{50} = 23.21 min.

C_v @ T_{50}
0.01 ft.^{2}/day

C_α = 0.021



Load No.= 12

Load= 4.00 ksf

D_0 = 0.34975

D_{50} = 0.34477

D_{100} = 0.33979

T_{50} = 18.71 min.

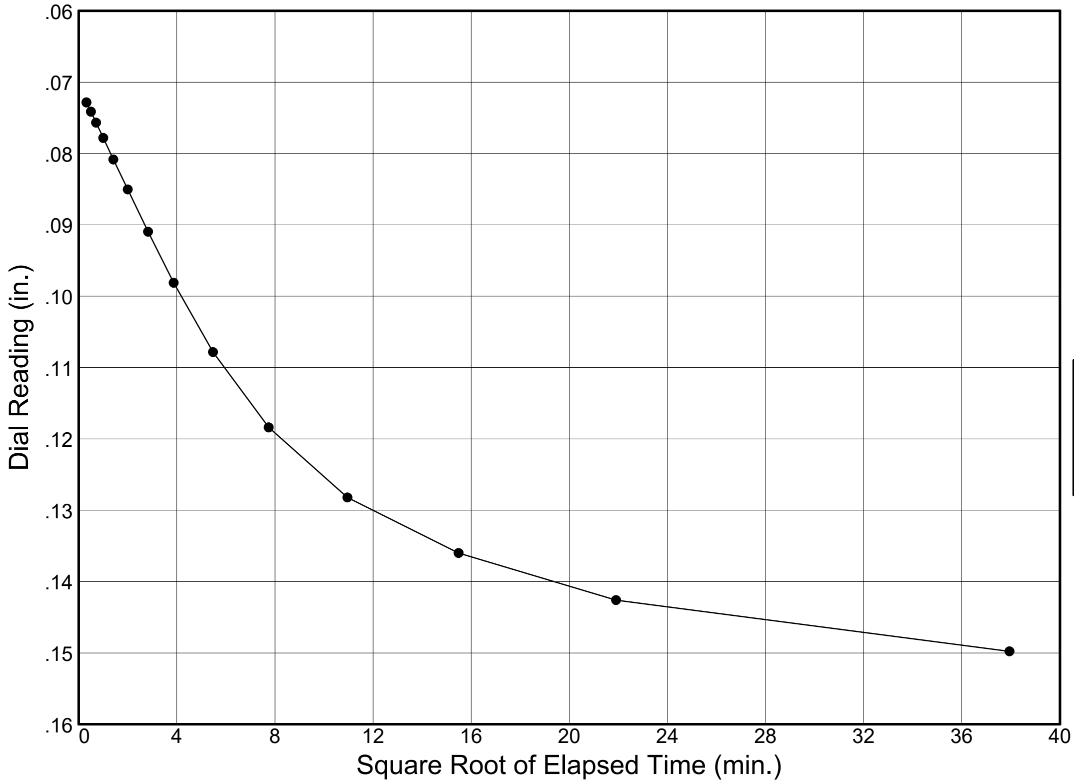
C_v @ T_{50}
0.01 ft.^{2}/day

ENGEO, INC.

Dial Reading vs. Time

Project No.: 9769.000.000
Project: Encinal Terminals

Source: B1-6 Sample No.: B1-6 @ 16.5



Load No.= 6

Load= 4.00 ksf

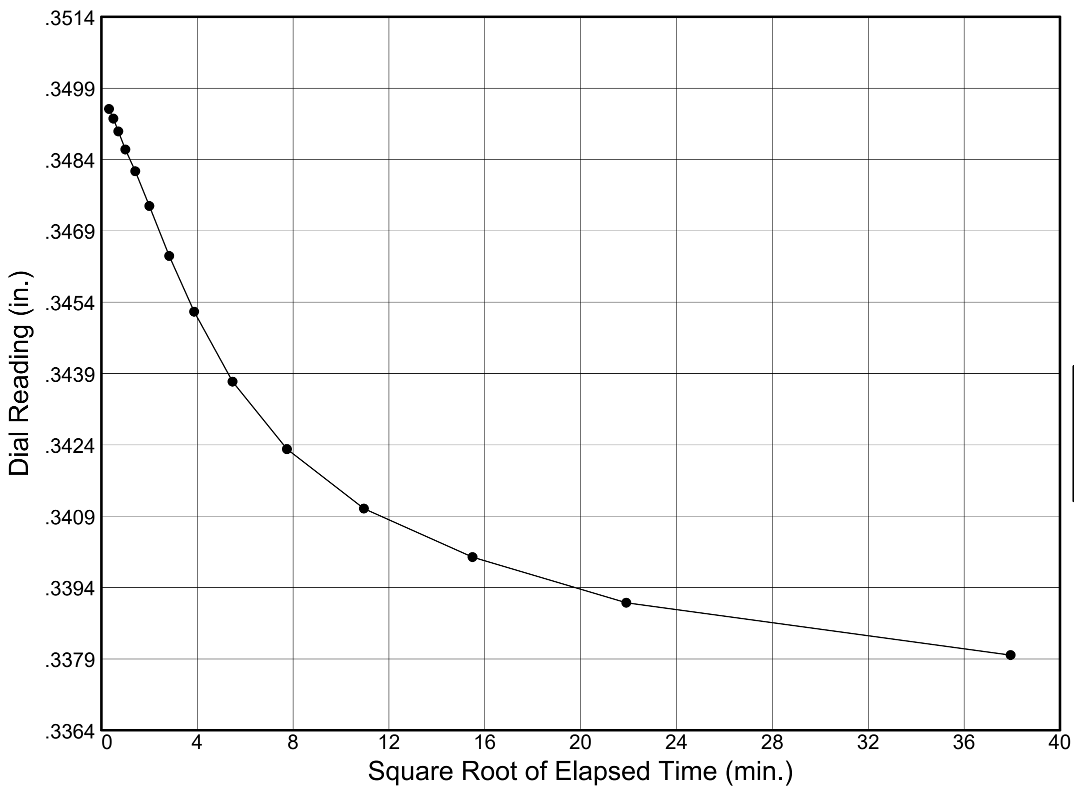
D_0 = 0.07185

D_{90} = 0.12348

D_{100} = 0.12921

T_{90} = 88.45 min.

C_v @ T_{90}
0.01 ft.^{2}/day



Load No.= 12

Load= 4.00 ksf

D_0 = 0.34975

D_{90} = 0.34267

D_{100} = 0.34189

T_{90} = 51.32 min.

C_v @ T_{90}
0.01 ft.^{2}/day

ENGEO, INC.

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	B1-4@31.5	**N**	31.5-32	2	**430**
2	B1-4@36.5	**N**	36.5-37	1	**347**
3	B1-5@16.5	**N**	16.5-17	3	**1966**
4	B1-5@21.5	**N**	21.5-22	1	**757**
5	B1-5@31.5	**N**	31.5-32	1	**551**
6	B1-5@36.5	**N**	36.5-37	4	**593**

PROJECT NAME: Encinal Terminals **DATE:** 2.5.13
PROJECT NUMBER: 9769.000.000
CLIENT: STL Company, LLC
PHASE NUMBER: 3

Tested by: **JL**
Reviewed by: **DS**

ENGEO INCORPORATED

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	B1-4@31.5	**N**	33-33.5	2	**430**
2	B1-4@36.5	**N**	38-38.5	1	**347**
3	B1-5@16.5	**N**	18-18.5	3	**1966**
4	B1-5@21.5	**N**	23-23.5	1	**757**
5	B1-5@31.5	**N**	34-34.5	1	**551**
6	B1-5@36.5	**N**	38.5-39	4	**593**

PROJECT NAME: Encinal Terminals **DATE:** 2.5.13
PROJECT NUMBER: 9769.000.000
CLIENT: STL Company, LLC
PHASE NUMBER: 3

Tested by: **JL**
Reviewed by: **DS**

ENGEO INCORPORATED

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
7	B1-5 @ 41.5	**N**	41.5-42	2	**520**
8	B1-5 @ 46.5	**N**	46.5-47	1	**418**
9	B1-5 @ 56.5	**N**	56.5-57	1	**66**
10	B1-6 @ 11.5	**N**	11.5-12	4	**1297**
11	B1-6 @ 16.5	**N**	16.5-17	2	**854**
12	B1-6 @ 21.5	**N**	21.5-22	3	**607**

Testing remarks: Sample #9 was very soft and saturated.

PROJECT NAME: Encinal Terminals
PROJECT NUMBER: 9769.000.000
CLIENT: STL Company, LLC
PHASE NUMBER: 003

DATE: 02/08/13

Tested by: JL
Reviewed by: GC

ENGEO INCORPORATED

EN GEO

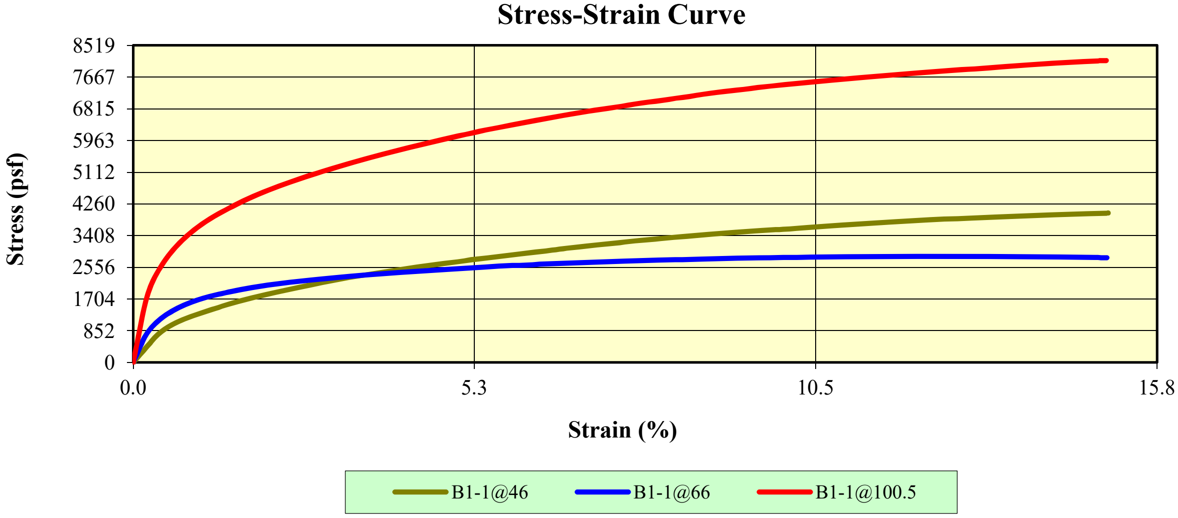
Unconsolidated Undrained Triaxial Test (ASTM D2850)

Project Information	
Project Name:	Encinal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Company, LLC
Boring Number:	B1-1
Sample Number:	Various
Sample Description:	See exploration logs

	B1-1@46	B1-1@66	B1-1@100.5	
Before Test				
Water Content (%)	22.1	44.3	19.2	
Dry Density (pcf)	109.4	78.5	113.5	
Saturation (%)	100.0	100.0	111.4	
Void Ratio	0.51	1.11	0.46	
Diameter (in)	2.38	2.39	2.39	
Height (in)	5.01	5.00	4.99	
Liquid Limit	--	--	--	
Plastic Limit	--	--	--	
Specific Gravity	2.315	2.506	2.379	
After Test				
Water Content (%)	22.1	44.3	19.2	
Saturation (%)	100.00	100.00	99.98	
Test Data				
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	4,014	2,852	8,114	
Cell Pressure (psf)	2,506	3,499	6,005	
At Failure				
σ_1 (psf)	6,520	6,352	14,118	
σ_3 (psf)	2,506	3,499	6,005	
Axial Strain @ Failure (%)	15.1	11.7	15.0	
Cohesion, c (psf)	2007	1426	4057	

Remarks: Cohesion is based on a horizontal line tangent with the respective Mohr circle.

Stress-Strain Curve



Tested By: D. Seibold Date: 2.7.13 Checked By: G. Criste Date

EN GEO

Unconsolidated Undrained Triaxial Test (ASTM D2850)

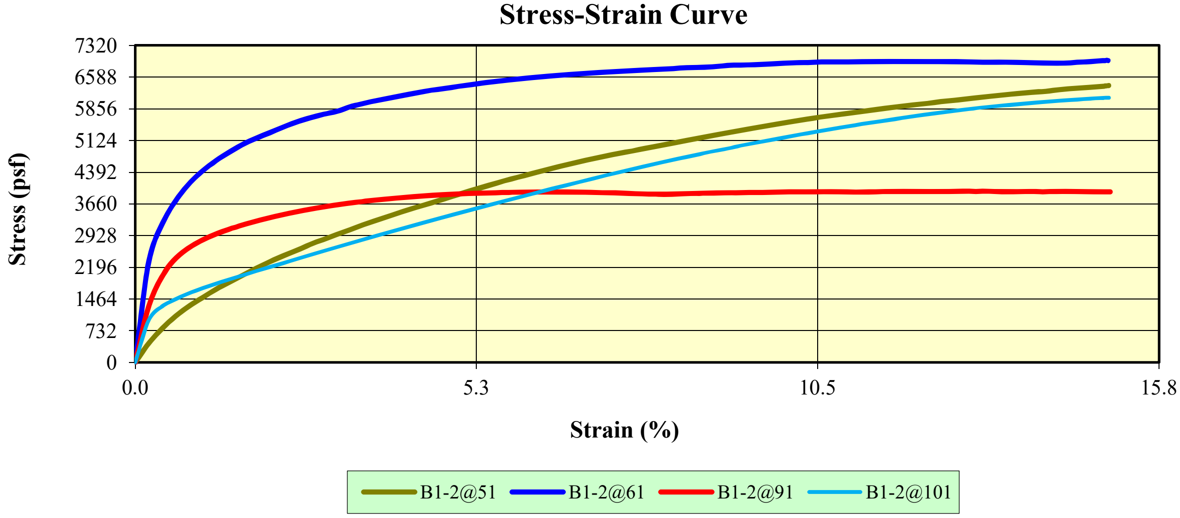
Tested By: G. Criste Date: 2.15.13 Checked By: D. Seibold Date 2.15.13

Project Information	
Project Name:	Encinal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Company, LLC
Boring Number:	B1-2
Sample Number:	Various
Sample Description:	See exploration logs

	B1-2@51	B1-2@61	B1-2@91	B1-2@101
Before Test				
Water Content (%)	27.3	22.6	33.4	23.5
Dry Density (pcf)	121.2	104.9	87.9	0.0
Saturation (%)	197.6	103.6	100.4	103.7
Void Ratio	0.37	0.58	0.88	0.60
Diameter (in)	2.40	2.40	2.40	0.00
Height (in)	5.04	5.03	5.07	0.00
Liquid Limit	--	--	--	
Plastic Limit	--	--	--	
Specific Gravity	2.650	2.650	2.650	2.650
After Test				
Water Content (%)	27.3	22.6	33.4	23.5
Saturation (%)	100.00	100.00	100.00	100.00
Test Data				
Strain Rate (in/min)	0.05	0.05	0.05	0.00
Peak Deviator Stress (psf)	6,393	6,971	3,952	6,112
Cell Pressure (psf)	2,995	3,499	5,501	6,005
At Failure				
σ_1 (psf)	9,388	10,471	9,453	12,117
σ_3 (psf)	2,995	3,499	5,501	6,005
Axial Strain @ Failure (%)	15.0	15.0	12.8	12.8
Cohesion, c (psf)	3196	3486	1976	3056

Remarks: Cohesion is based on a horizontal line tangent to the respective Mohr circle

Stress-Strain Curve



Tested By: D. Seibold Date: 2.12.13 Checked By: G. Criste Date 2.12.13

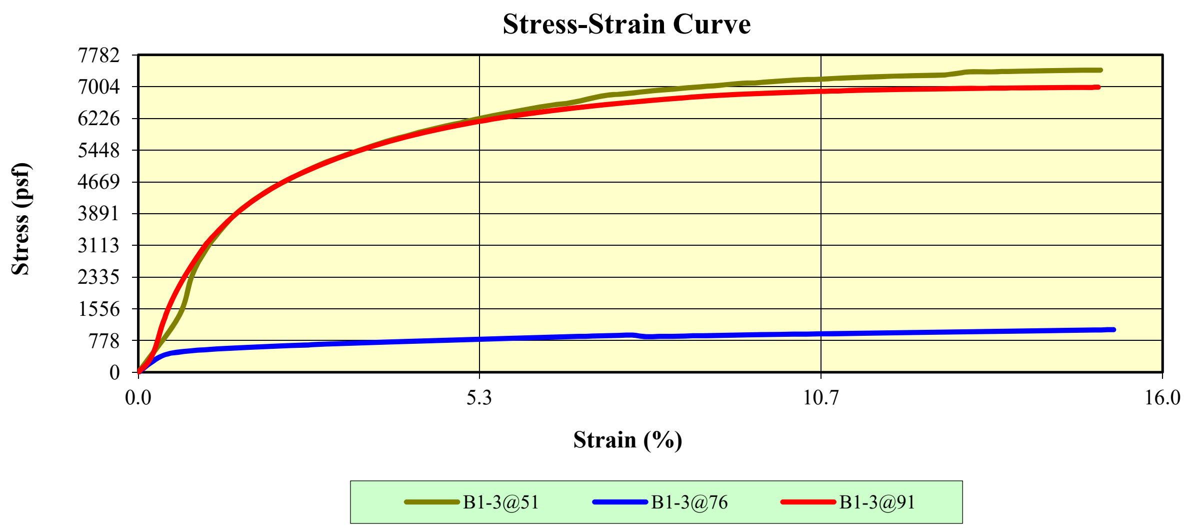
EN GEO

Unconsolidated Undrained Triaxial Test (ASTM D2850)

Project Information	
Project Name:	Encinal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Company, LLC
Boring Number:	B1-3
Sample Number:	Various
Sample Description:	See exploration logs

	B1-3@51	**B1-3@76**	**B1-3@91**	
Before Test				
Water Content (%)	22.3	58.4	21.6	
Dry Density (pcf)	107.9	66.6	108.9	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.53	1.48	0.52	
Diameter (in)	2.40	2.39	2.39	
Height (in)	4.99	4.99	5.01	
Liquid Limit	--	--	--	
Plastic Limit	--	--	--	
Specific Gravity	2.400	2.540	2.410	
After Test				
Water Content (%)	22.3	0.0	0.0	
Saturation (%)	100.00	0.00	0.00	
Test Data				
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	7,412	1,047	6,988	
Cell Pressure (psf)	2,506	4,003	5,501	
At Failure				
σ_1 (psf)	9,917	5,050	12,489	
σ_3 (psf)	2,506	4,003	5,501	
Axial Strain @ Failure (%)	15.1	15.3	15.0	
Cohesion, c (psf)	3706	523	3494	

Remarks: Cohesion is based on a horizontal line tangent with the respective Mohr circle



EN **GEO**

Unconsolidated Undrained Triaxial Test (ASTM D2850)

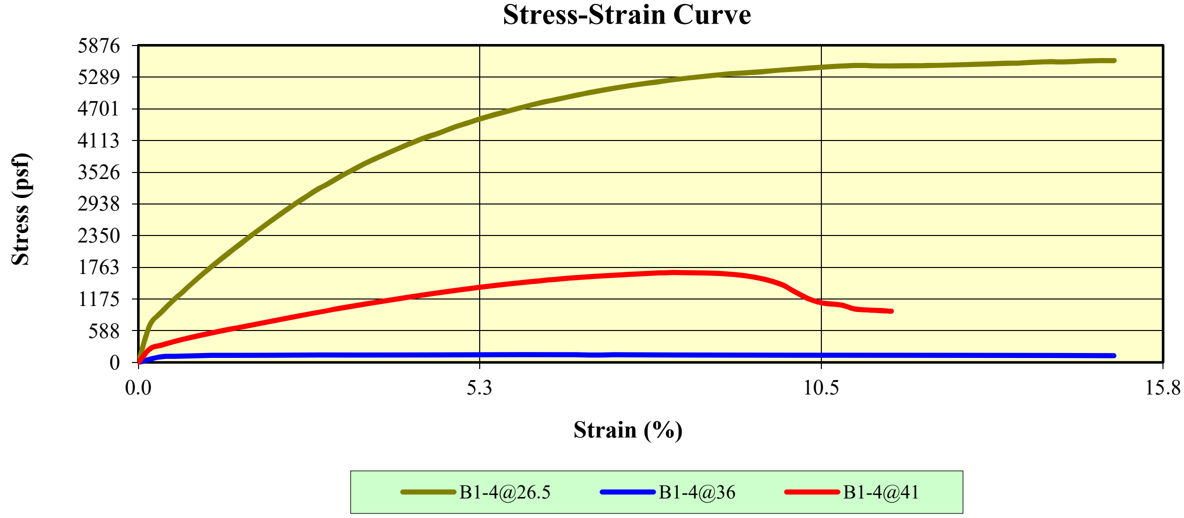
Date 02/15/13

Project Information	
Project Name:	Encimal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Companies, LLC
Boring Number:	B1-4
Sample Number:	Various
Sample Description:	See exploration logs

	B1-4@26.5	B1-4@36	B1-4@41	
Before Test				
Water Content (%)	17.8	116.5	226.3	
Dry Density (pcf)	114.7	39.8	24.0	
Saturation (%)	106.6	97.6	101.7	
Void Ratio	0.44	3.16	5.90	
Diameter (in)	2.86	2.87	2.87	
Height (in)	6.01	5.56	5.78	
Liquid Limit	--	--	--	
Plastic Limit	--	--	--	
Specific Gravity	2.650	2.650	2.650	
After Test				
Water Content (%)	17.8	116.5	226.3	
Saturation (%)	100.00	97.63	100.00	
Test Data				
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	5,596	145	1,666	
Cell Pressure (psf)	806	994	1,498	
At Failure				
σ_1 (psf)	6,403	1,138	3,163	
σ_3 (psf)	806	994	1,498	
Axial Strain @ Failure (%)	15.1	6.2	8.3	
Cohesion, c (psf)	2798	72	833	

Remarks: Cohesion is based on a horizontal line tangent to the respective Mohr circle

Stress-Strain Curve



Tested By: G. Criste Date: 02/14/13 Checked By: D. Seibold

EN **GEO**

Unconsolidated Undrained Triaxial Test (ASTM D2850)

Date

Checked By:

Date:

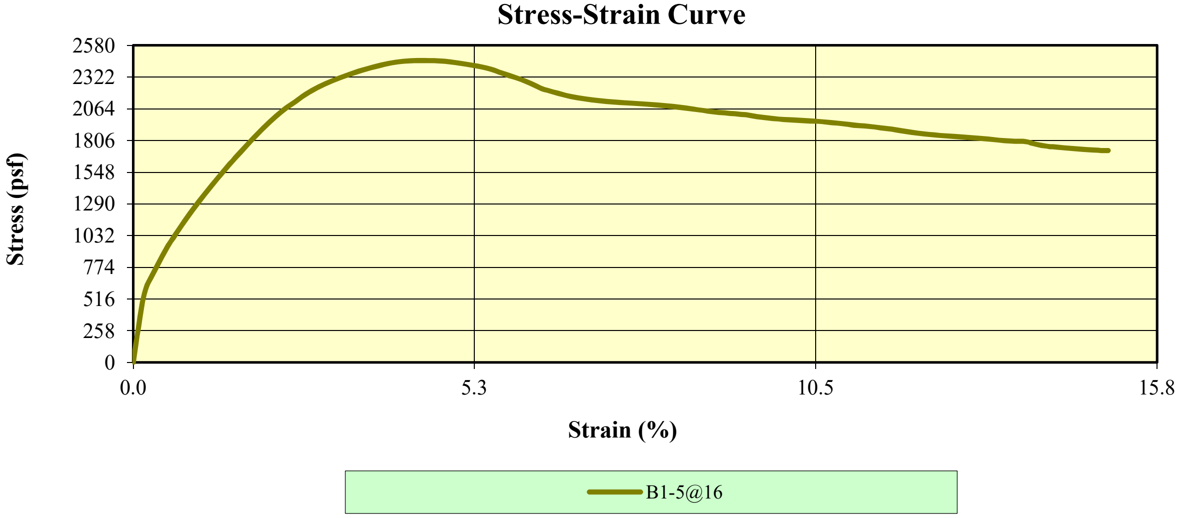
Tested By:

Project Information	
Project Name:	Encinal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Company LLC.
Boring Number:	B1-5
Sample Number:	Various
Sample Description:	See exploration logs

	B1-5@16			
Before Test				
Water Content (%)	65.9			
Dry Density (pcf)	59.9			
Saturation (%)	99.2			
Void Ratio	1.76			
Diameter (in)	2.87			
Height (in)	6.32			
Liquid Limit	--			
Plastic Limit	--			
Specific Gravity	2.650			
After Test				
Water Content (%)	65.9			
Saturation (%)	99.21			
Test Data				
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	2,457			
Cell Pressure (psf)	806			
At Failure				
σ_1 (psf)	3,263			
σ_3 (psf)	806			
Axial Strain @ Failure (%)	4.5			
Cohesion, c (psf)	1228			

Remarks: Cohesion is based on a horizontal line tangent with the respective Mohr circle

Stress-Strain Curve



9769000000_TxUU_B1-5.HSD

EN **GEO**

Unconsolidated Undrained Triaxial Test (ASTM D2850)

Date

Checked By:

Date:

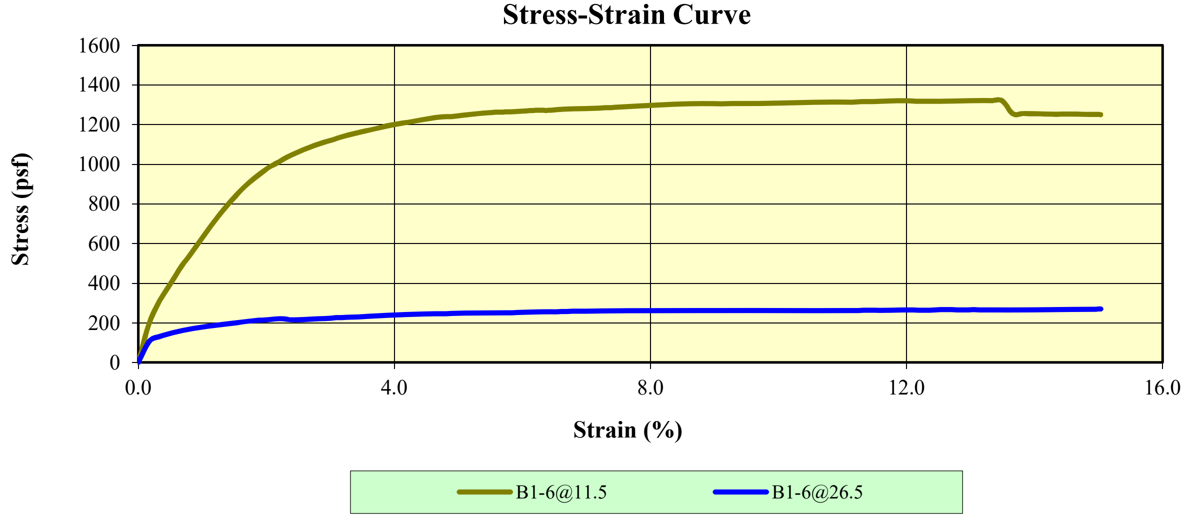
Tested By:

Project Information	
Project Name:	Encinal Terminals
Project Number:	9769.000.000
Location:	Alameda, California
Client:	STL Company, LLC
Boring Number:	B1-6
Sample Number:	Various
Sample Description:	See exploration logs

	B1-6@11	B1-6@26.5	Test 3	Test 4
Before Test				
Water Content (%)	52.5	32.3		
Dry Density (pcf)	67.4	86.4		
Saturation (%)	95.8	93.6		
Void Ratio	1.45	0.92		
Diameter (in)	2.86	2.85		
Height (in)	6.20	6.09		
Liquid Limit	--	--		
Plastic Limit	--	--		
Specific Gravity	2.650	2.650		
After Test				
Water Content (%)	52.5	32.3		
Saturation (%)	95.80	93.58		
Test Data				
Strain Rate (in/min)	0.05	0.05		
Peak Deviator Stress (psf)	1,322	270		
Cell Pressure (psf)	504	806		
At Failure				
σ_1 (psf)	1,826	1,076		
σ_3 (psf)	504	806		
Axial Strain @ Failure (%)	13.2	15.0		
Cohesion, c (psf)	661	135		

Remarks: Cohesion is based on a horizontal line tangent to the respective Mohr circle

Stress-Strain Curve



APPENDIX C

LIQUEFACTION ANALYSIS

APPENDIX C

ENGEO
Expect Excellence

ENGEO

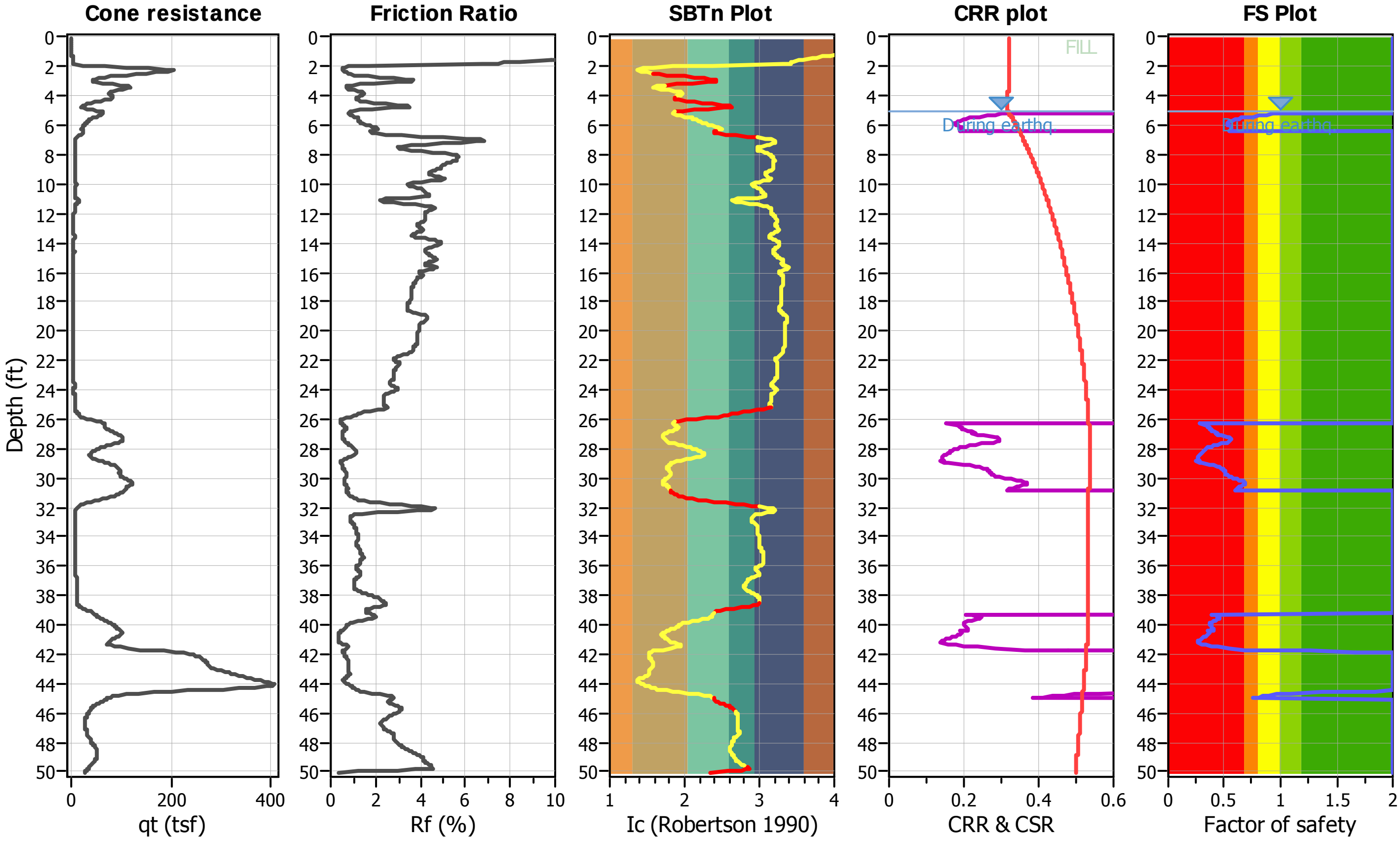
LIQUEFACTION ANALYSIS REPORT

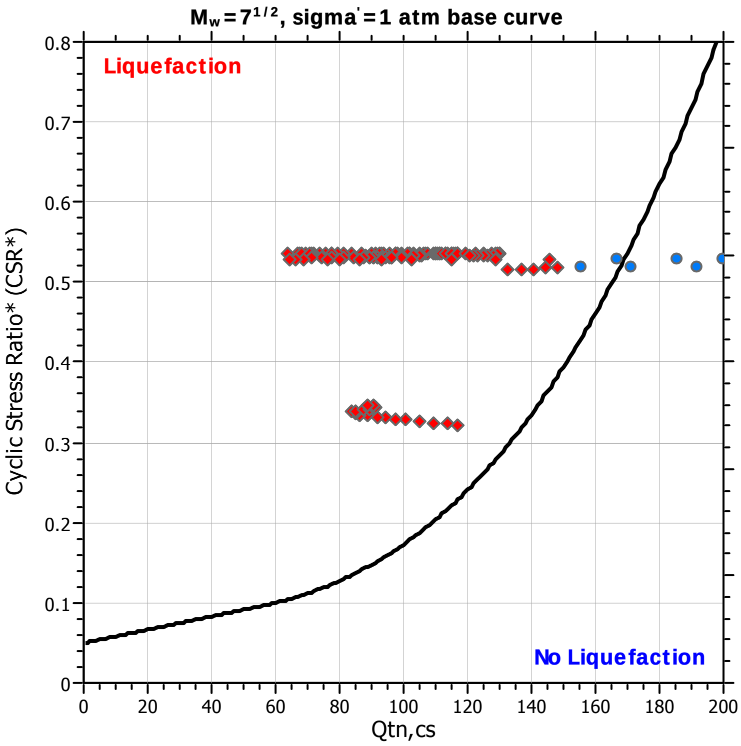
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

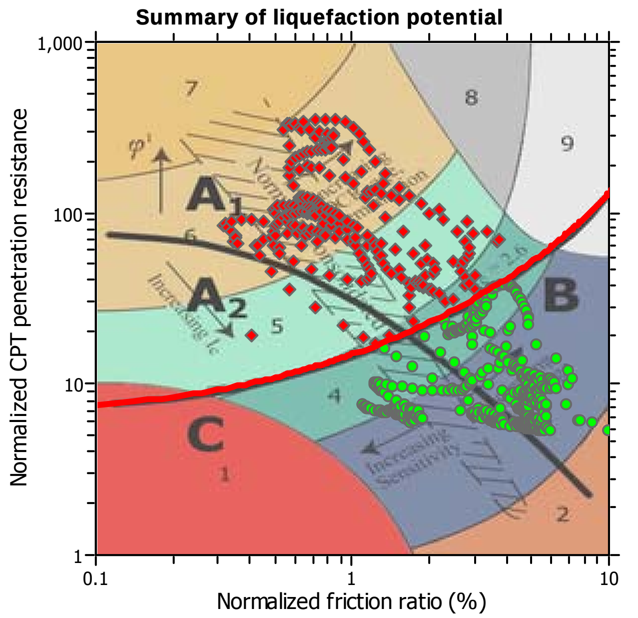
CPT file : 2-CPT01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

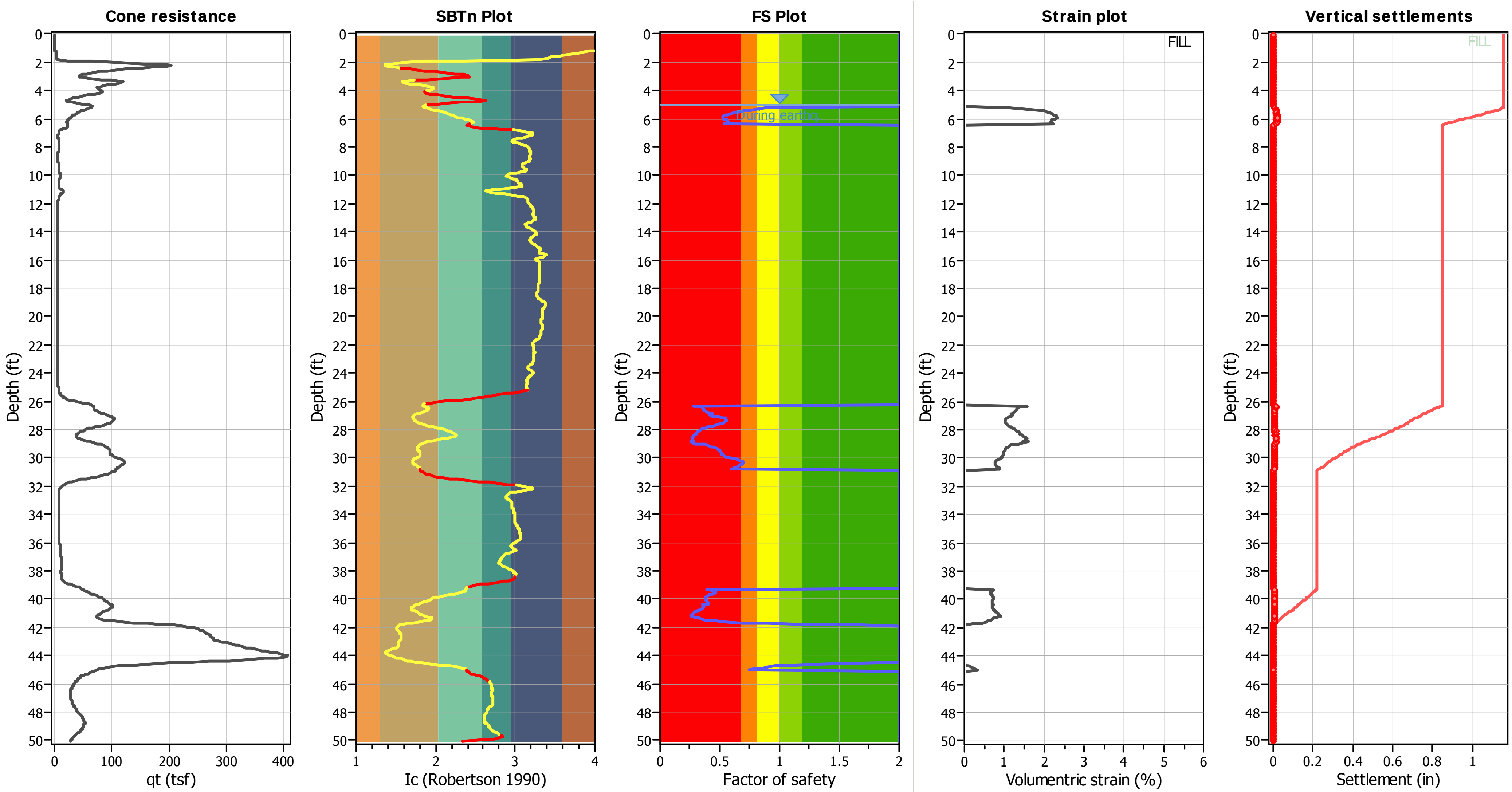






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

ENGEO Expect Excellence

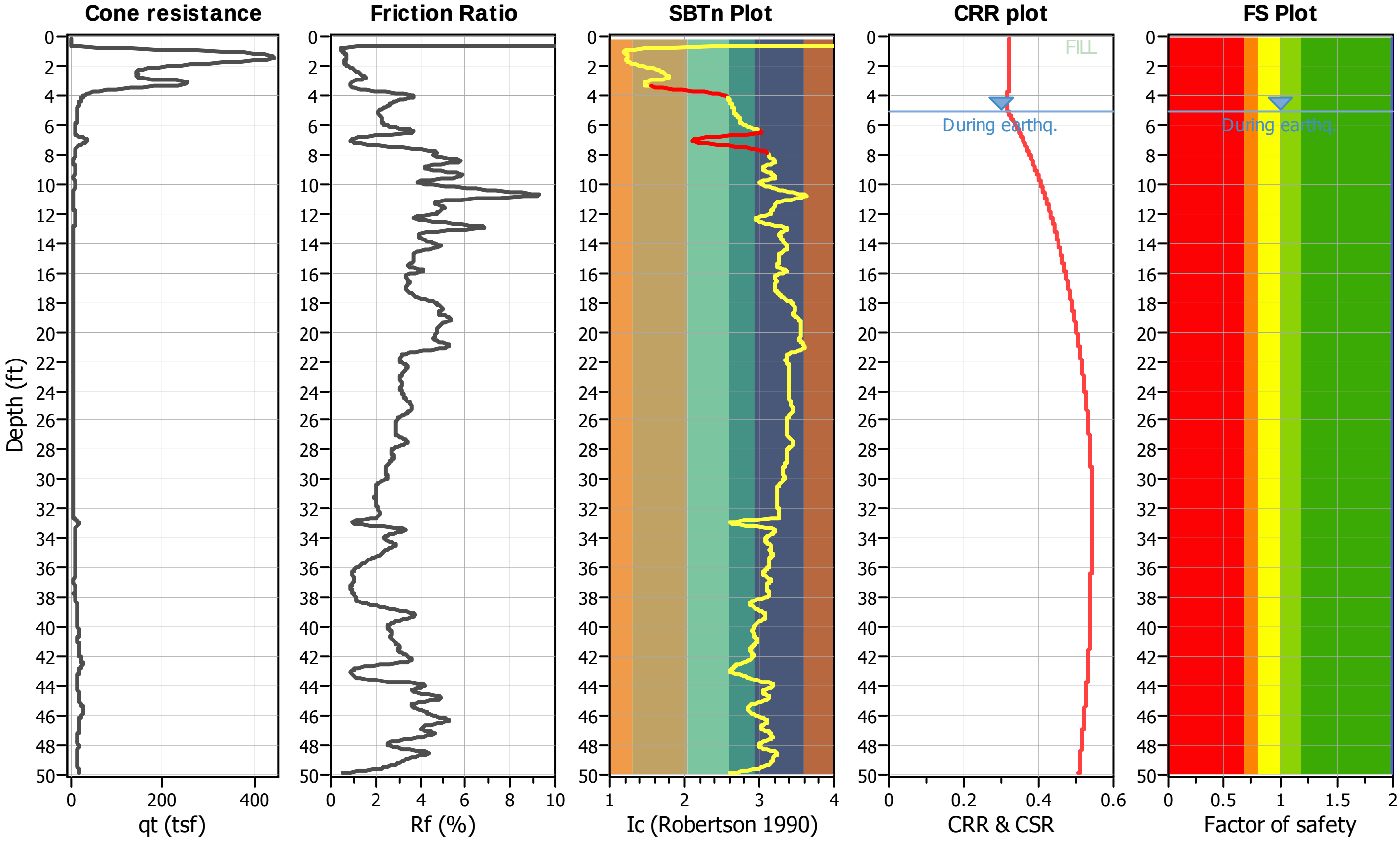
LIQUEFACTION ANALYSIS REPORT

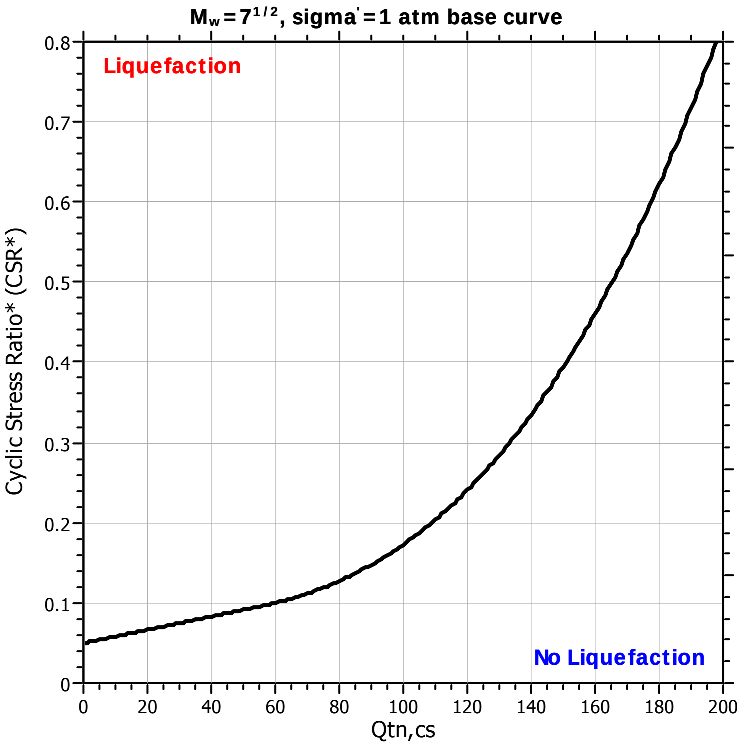
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

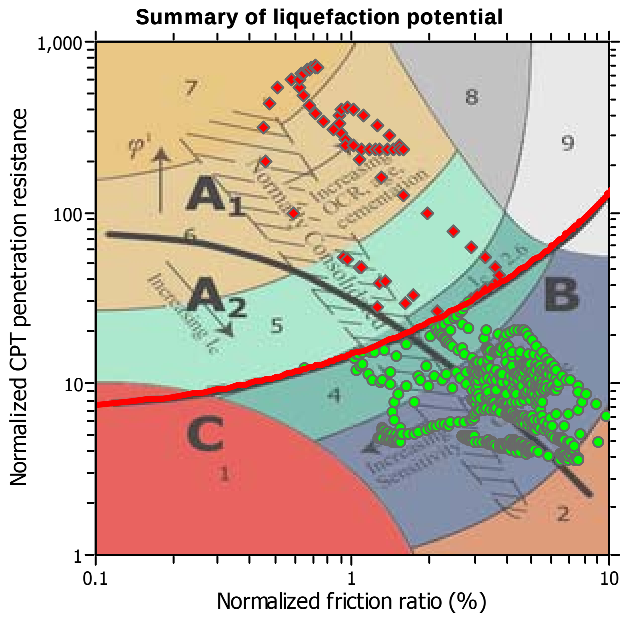
CPT file : 2-CPT02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







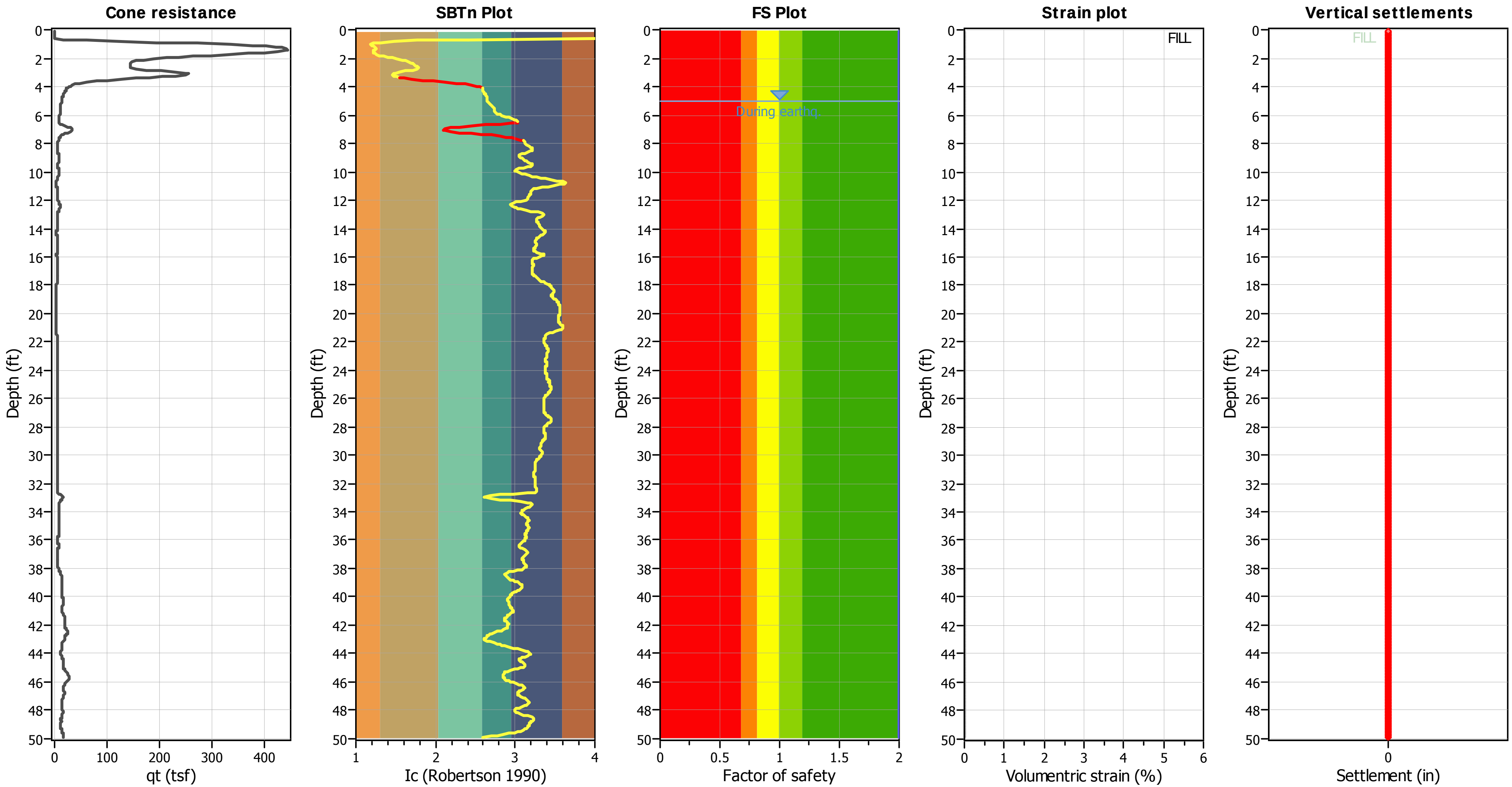
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

Expect Excellence

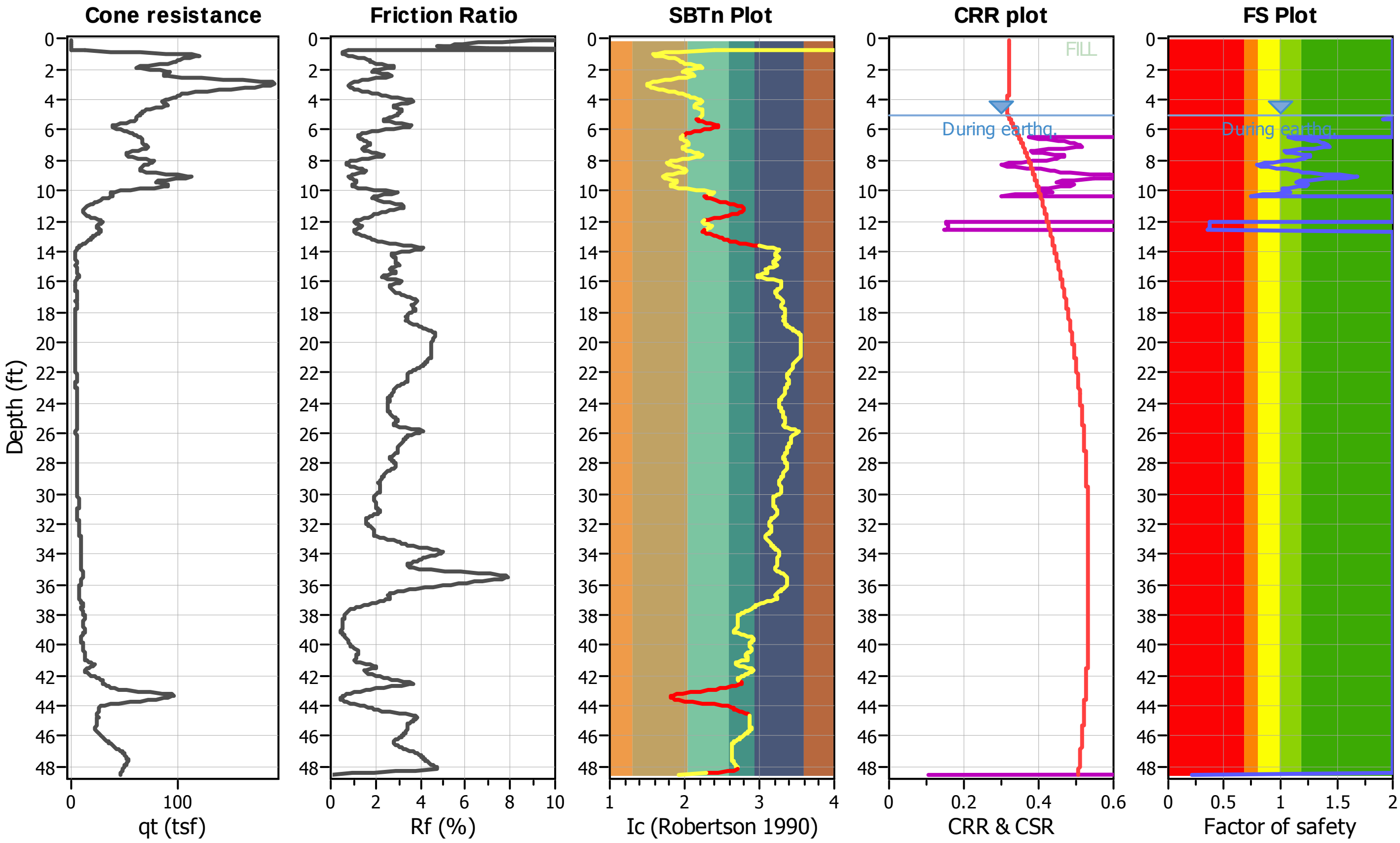
LIQUEFACTION ANALYSIS REPORT

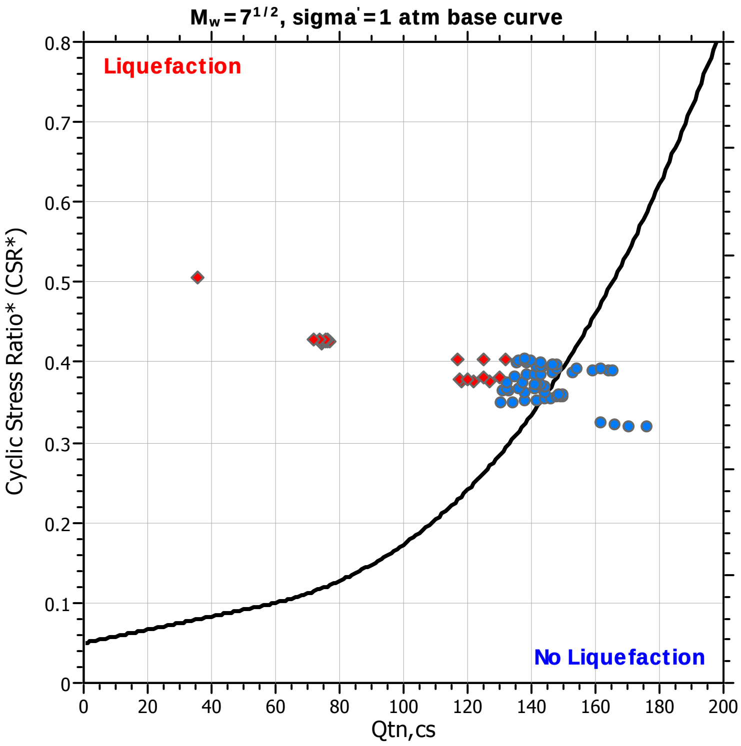
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

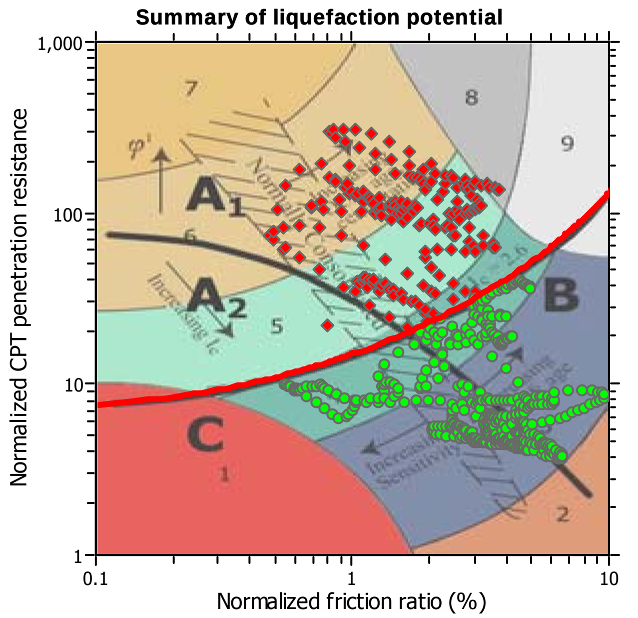
CPT file : 2-CPT03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







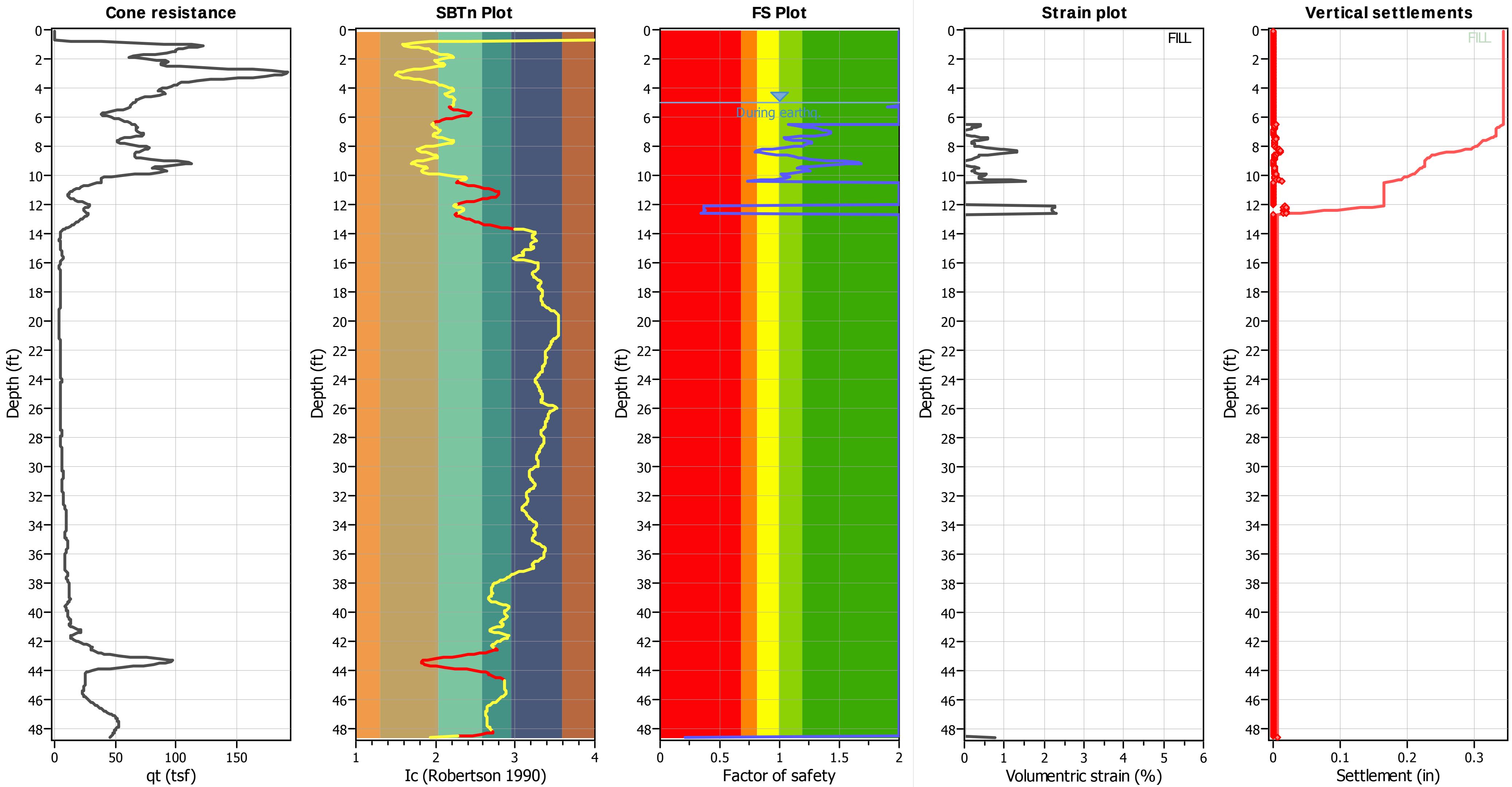
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

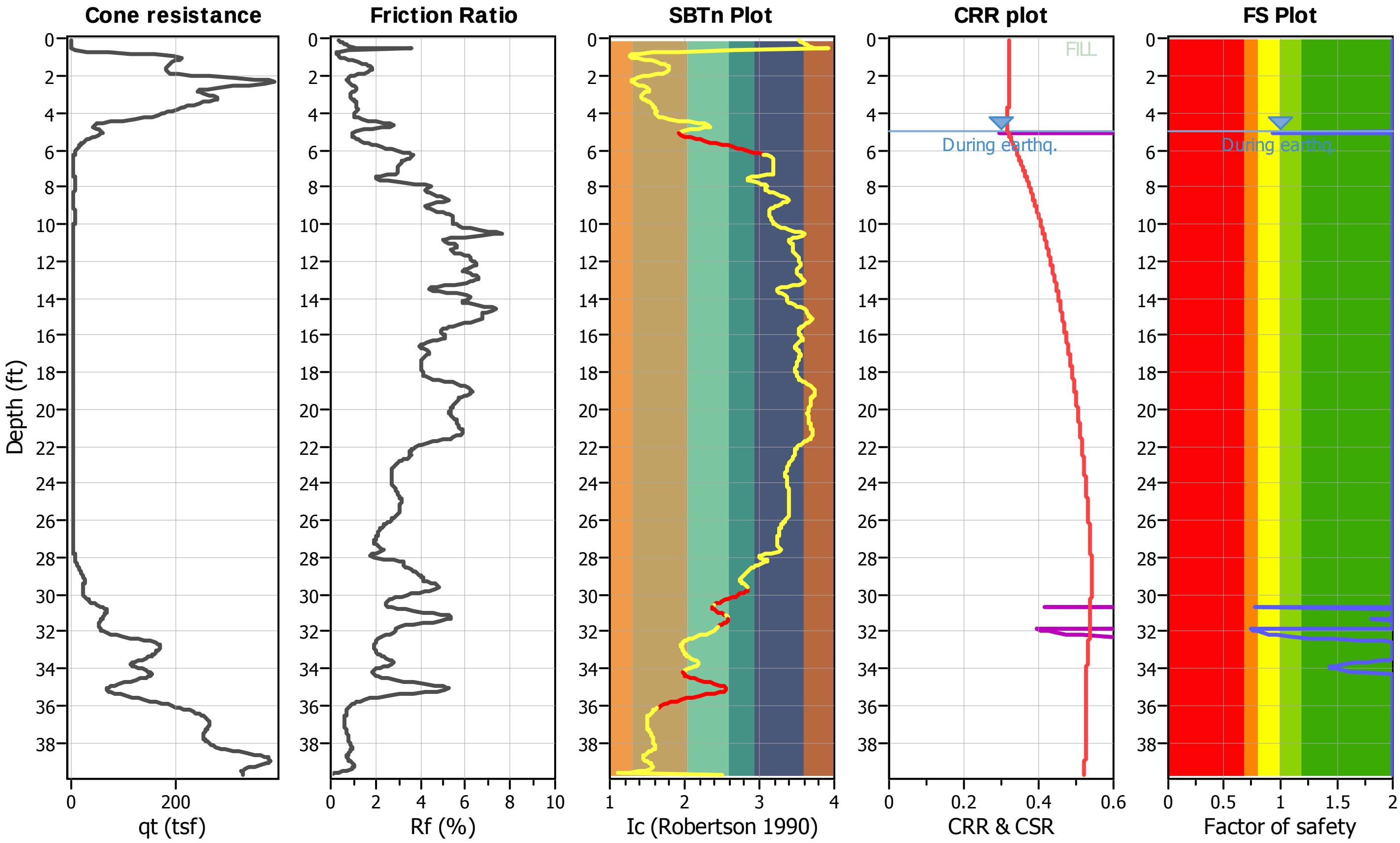
LIQUEFACTION ANALYSIS REPORT

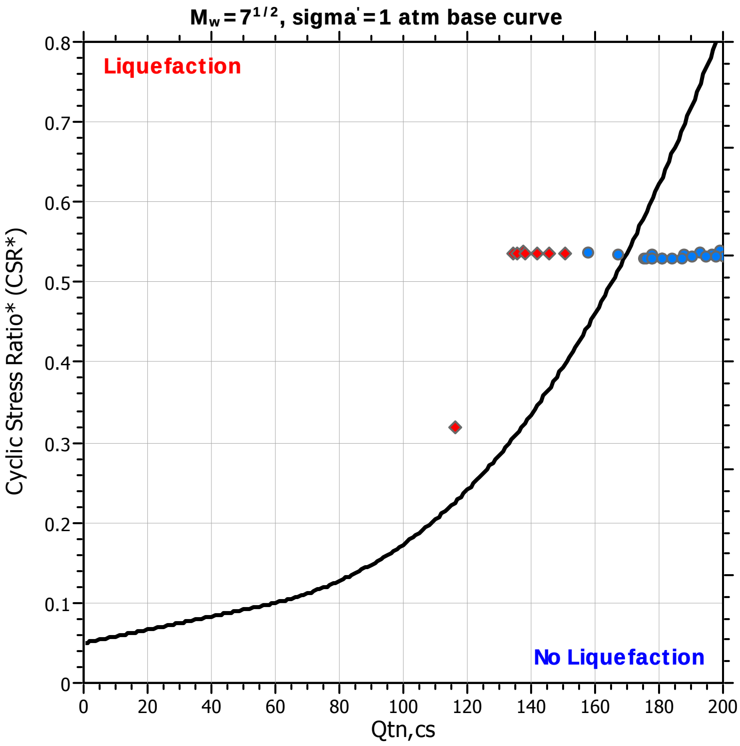
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

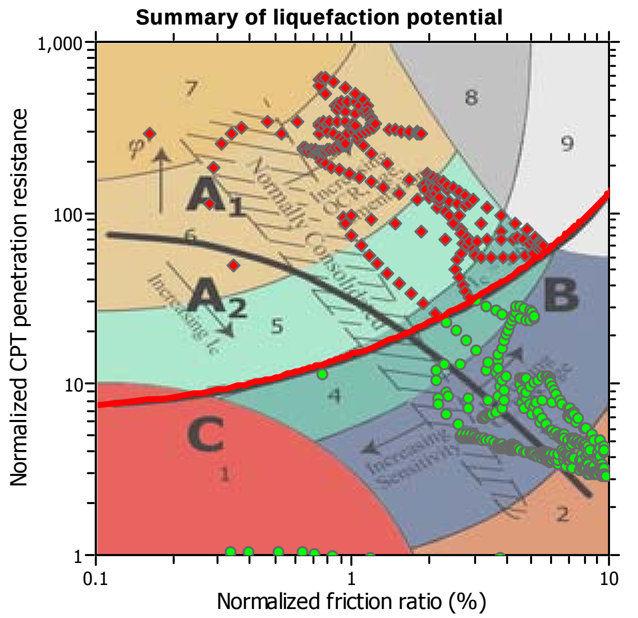
CPT file : 2-CPT04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

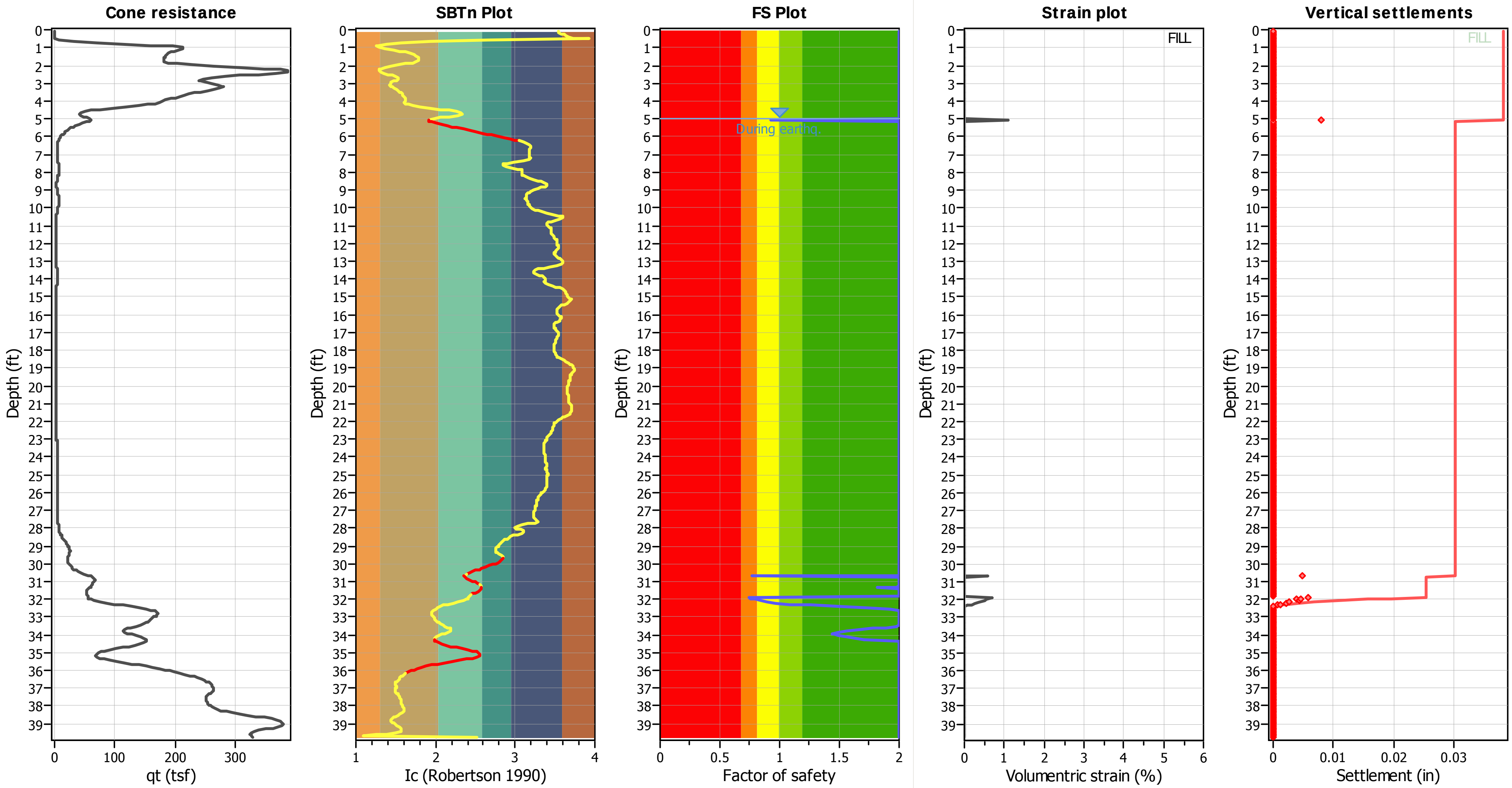






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

Expect Excellence

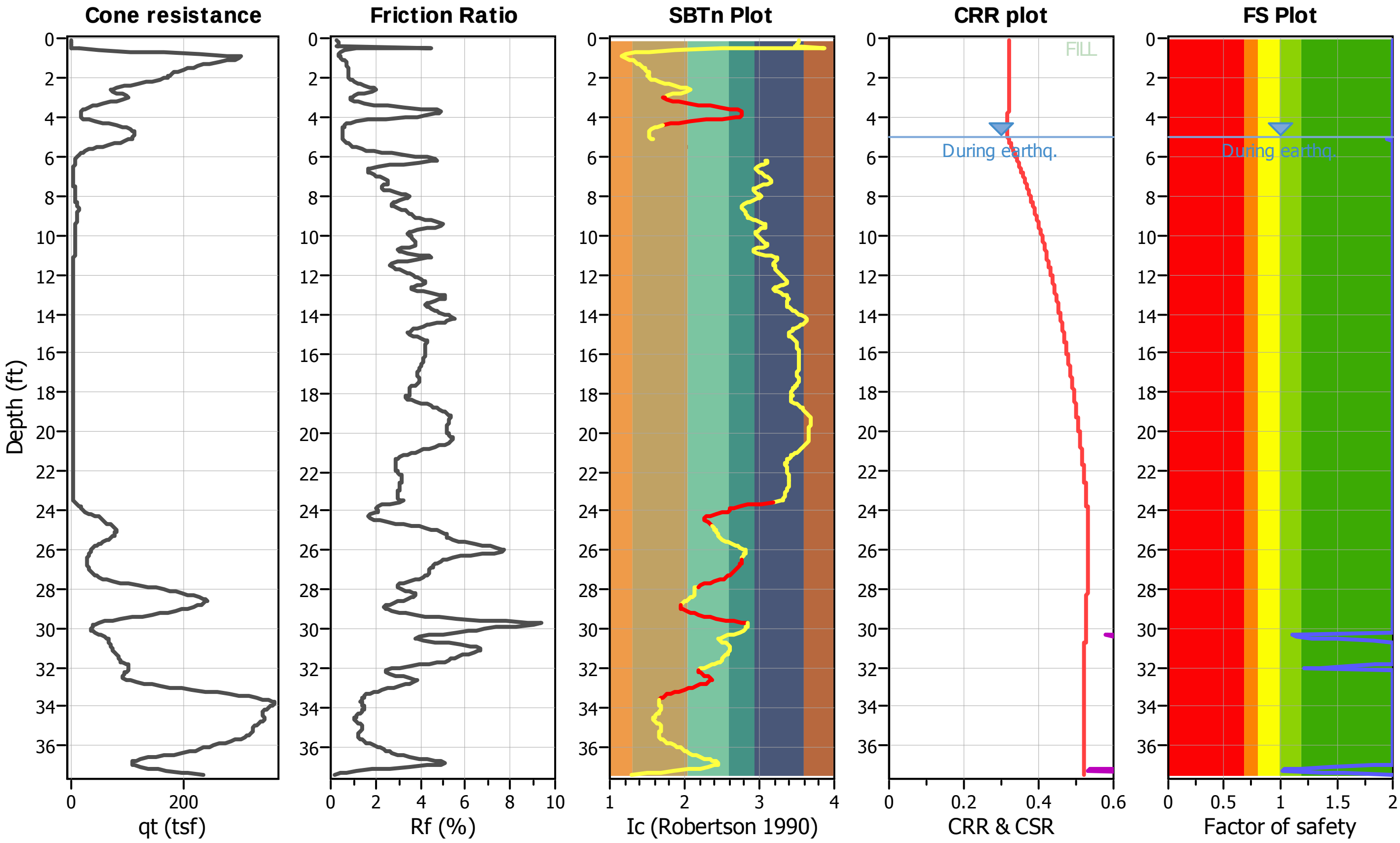
LIQUEFACTION ANALYSIS REPORT

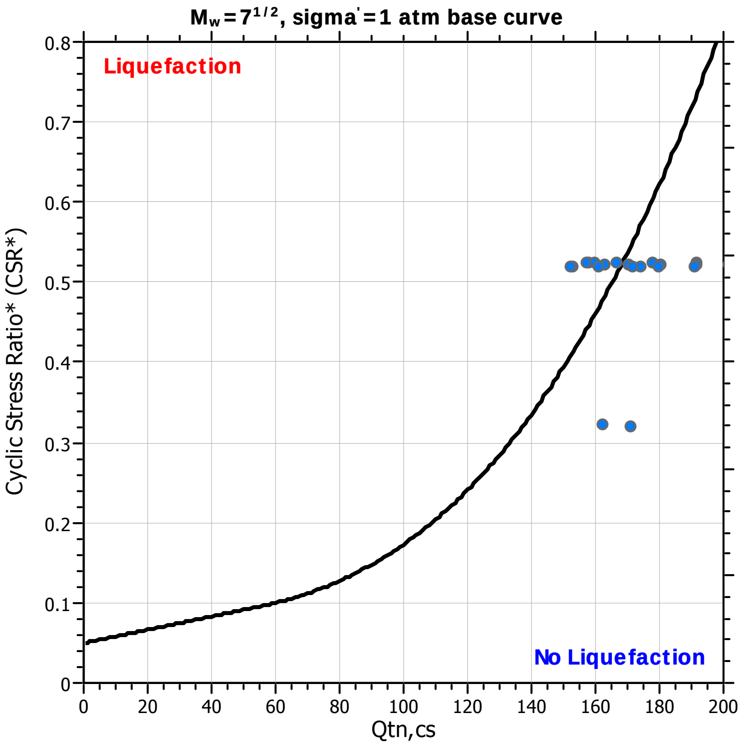
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

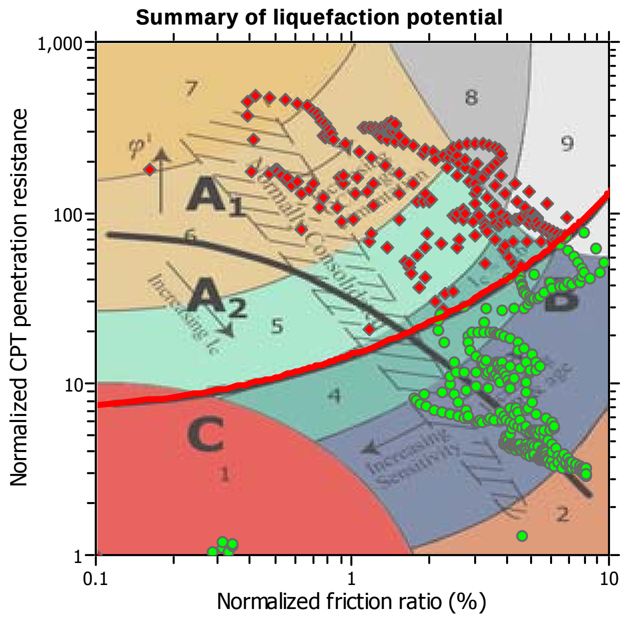
CPT file : 2-CPT05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







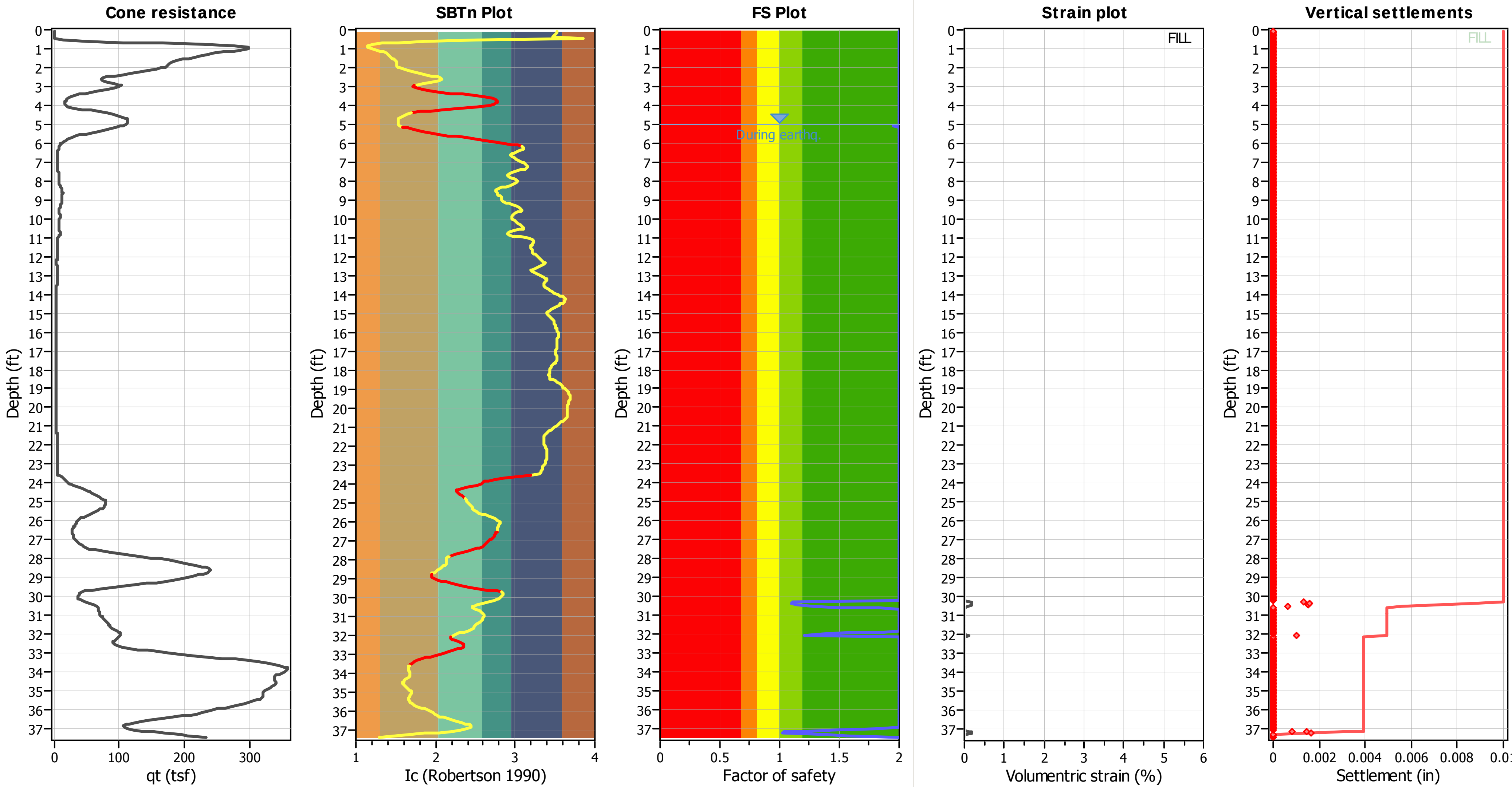
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

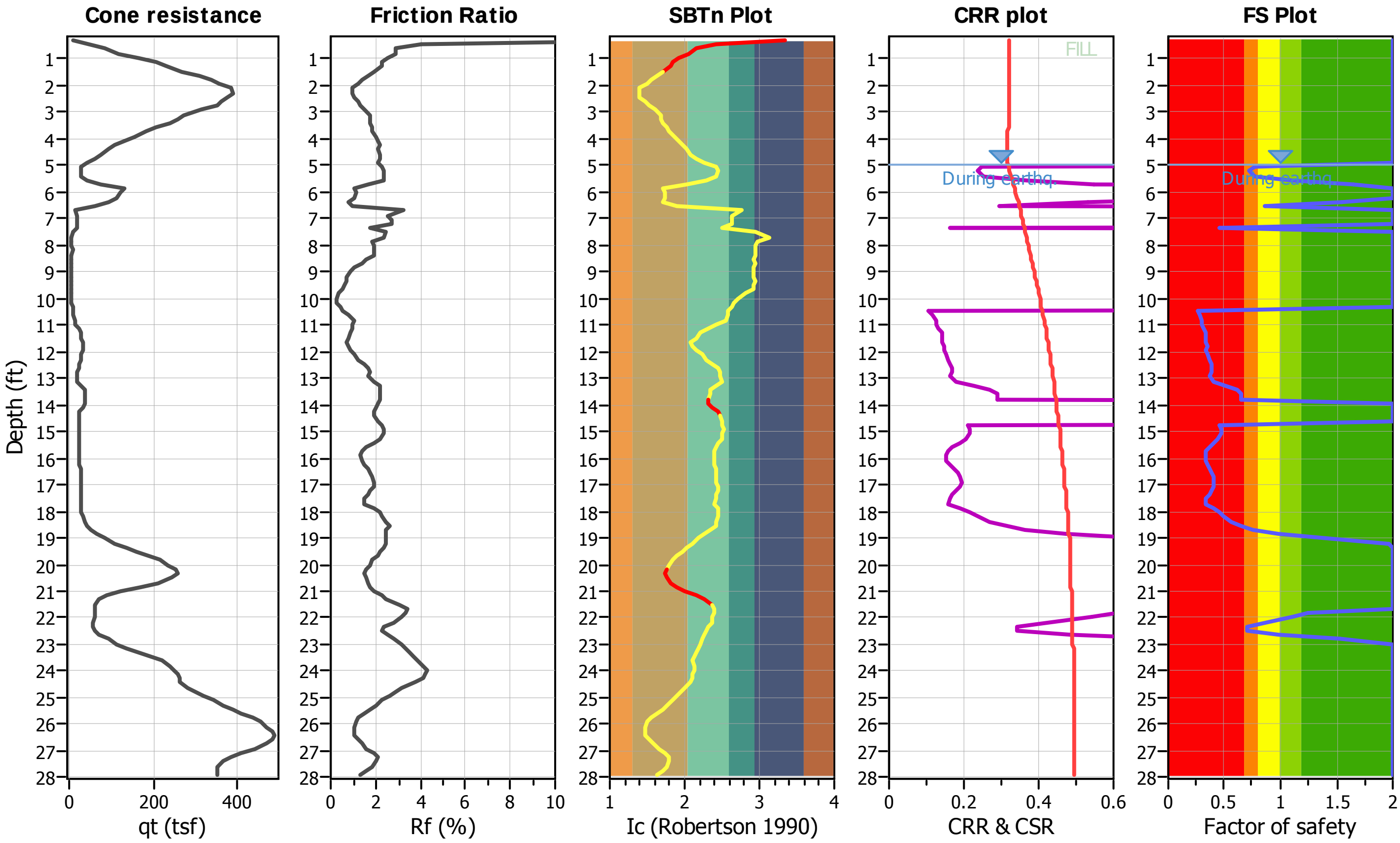
LIQUEFACTION ANALYSIS REPORT

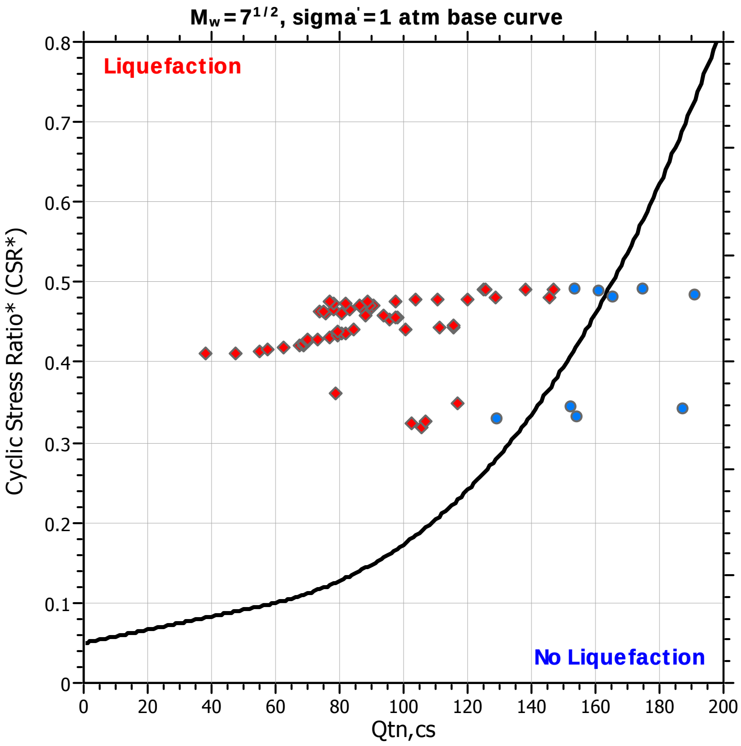
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

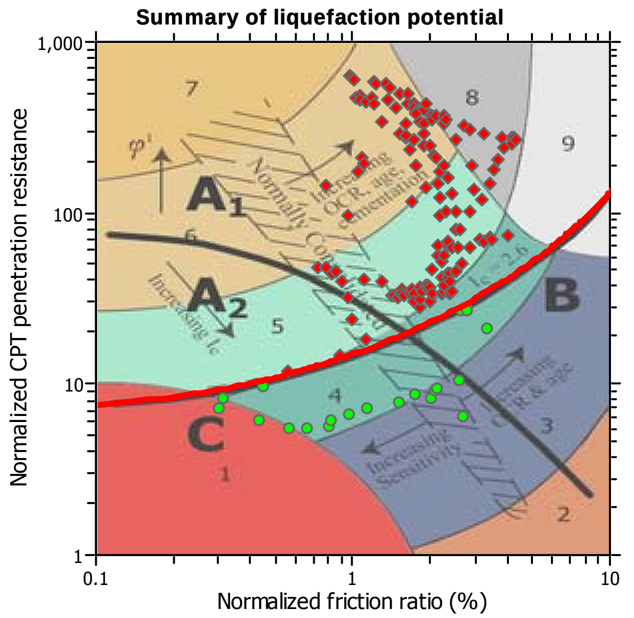
CPT file : 3-CPT01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

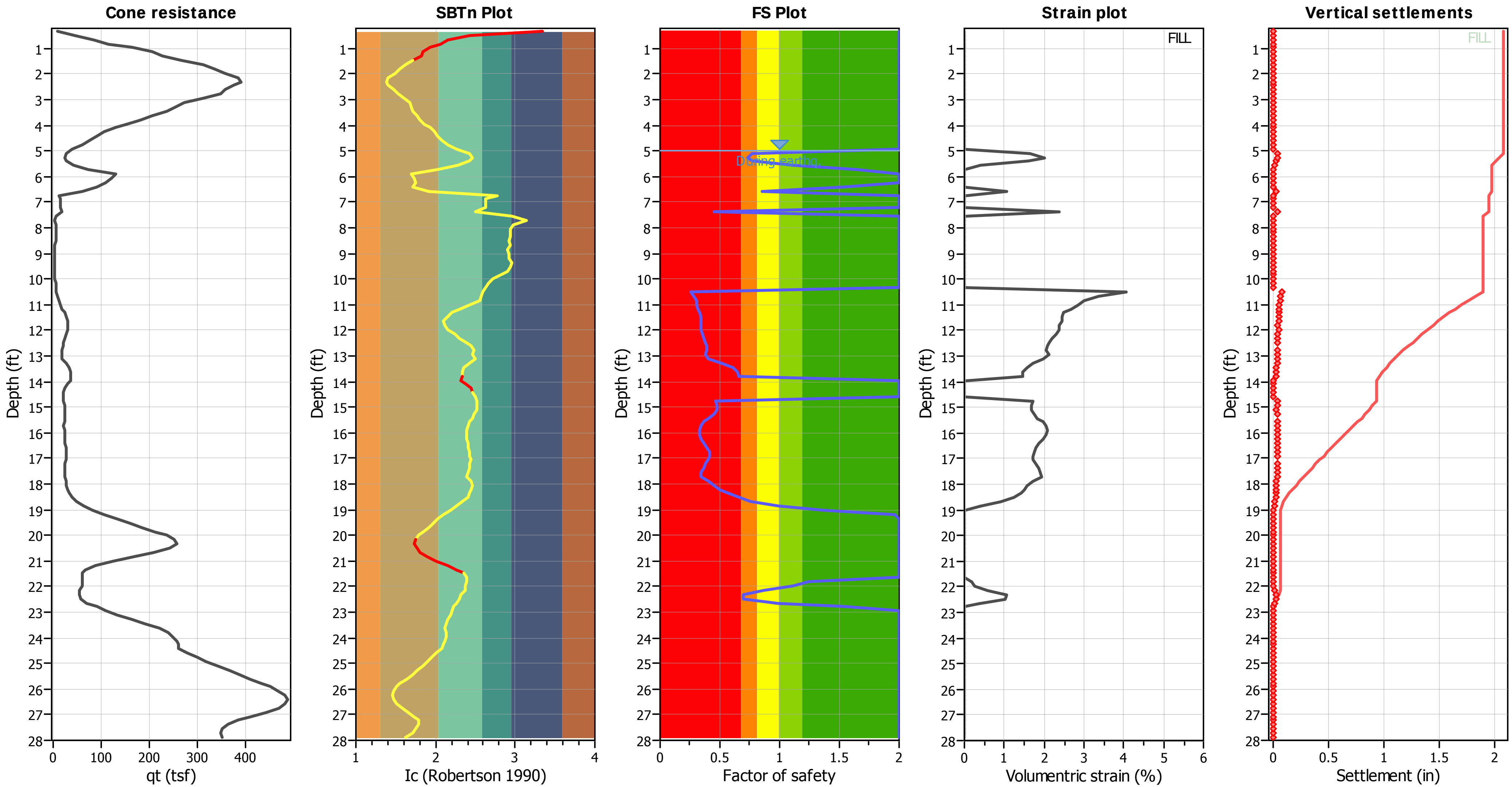






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

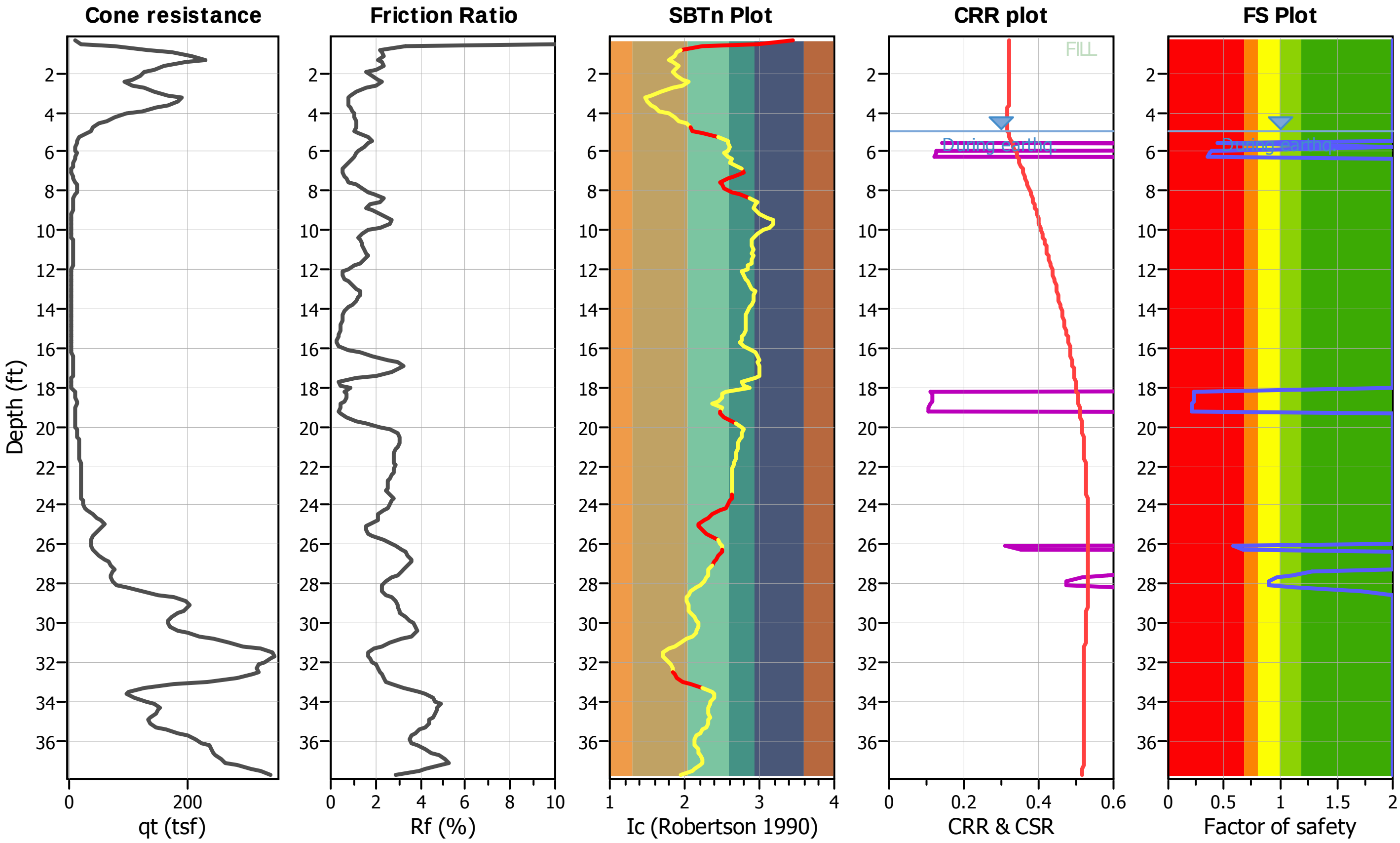
LIQUEFACTION ANALYSIS REPORT

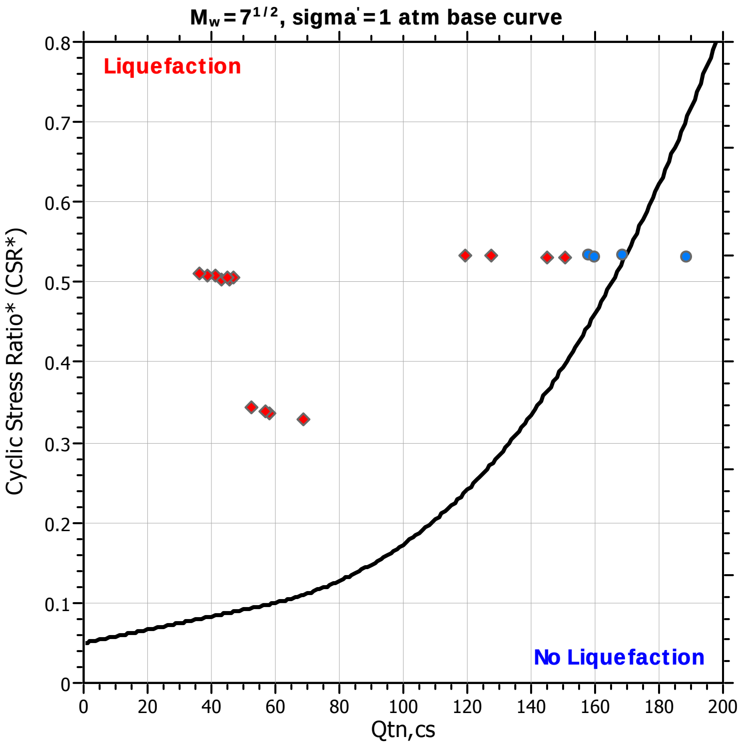
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

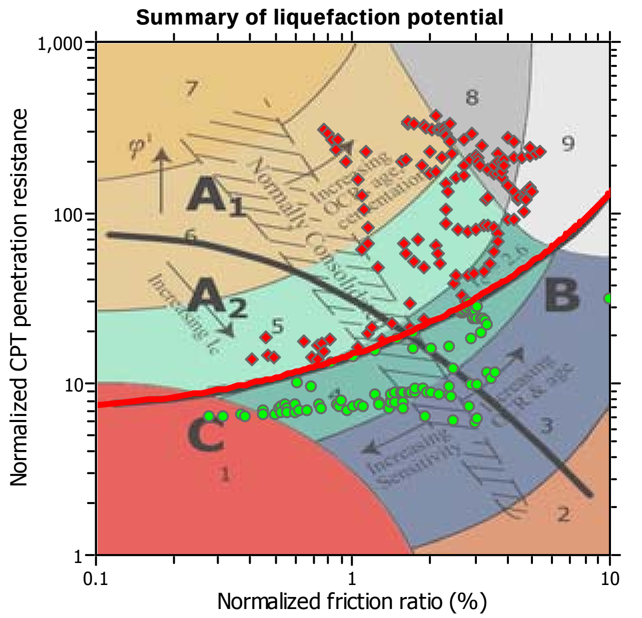
CPT file : 3-CPT02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

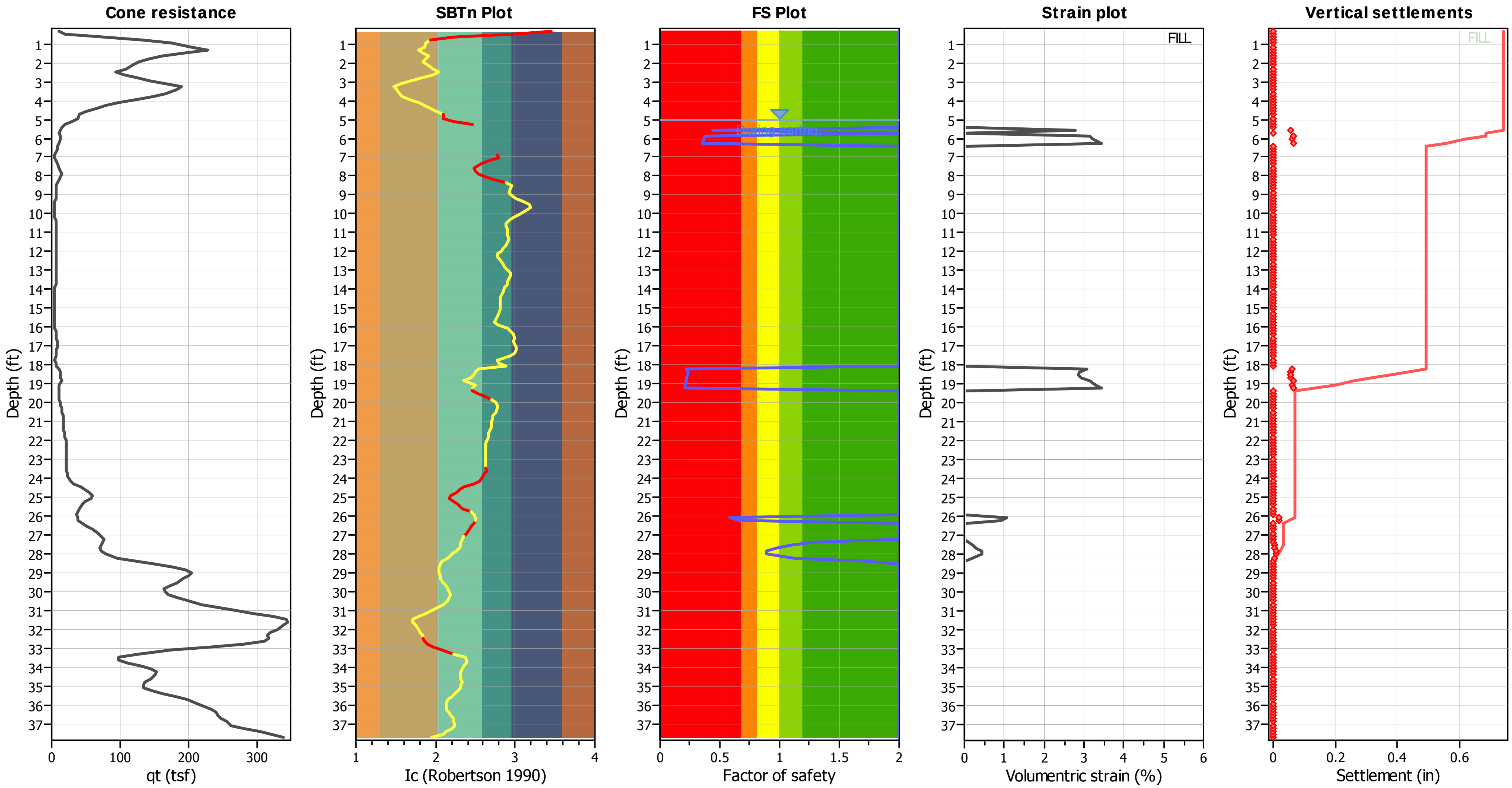






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

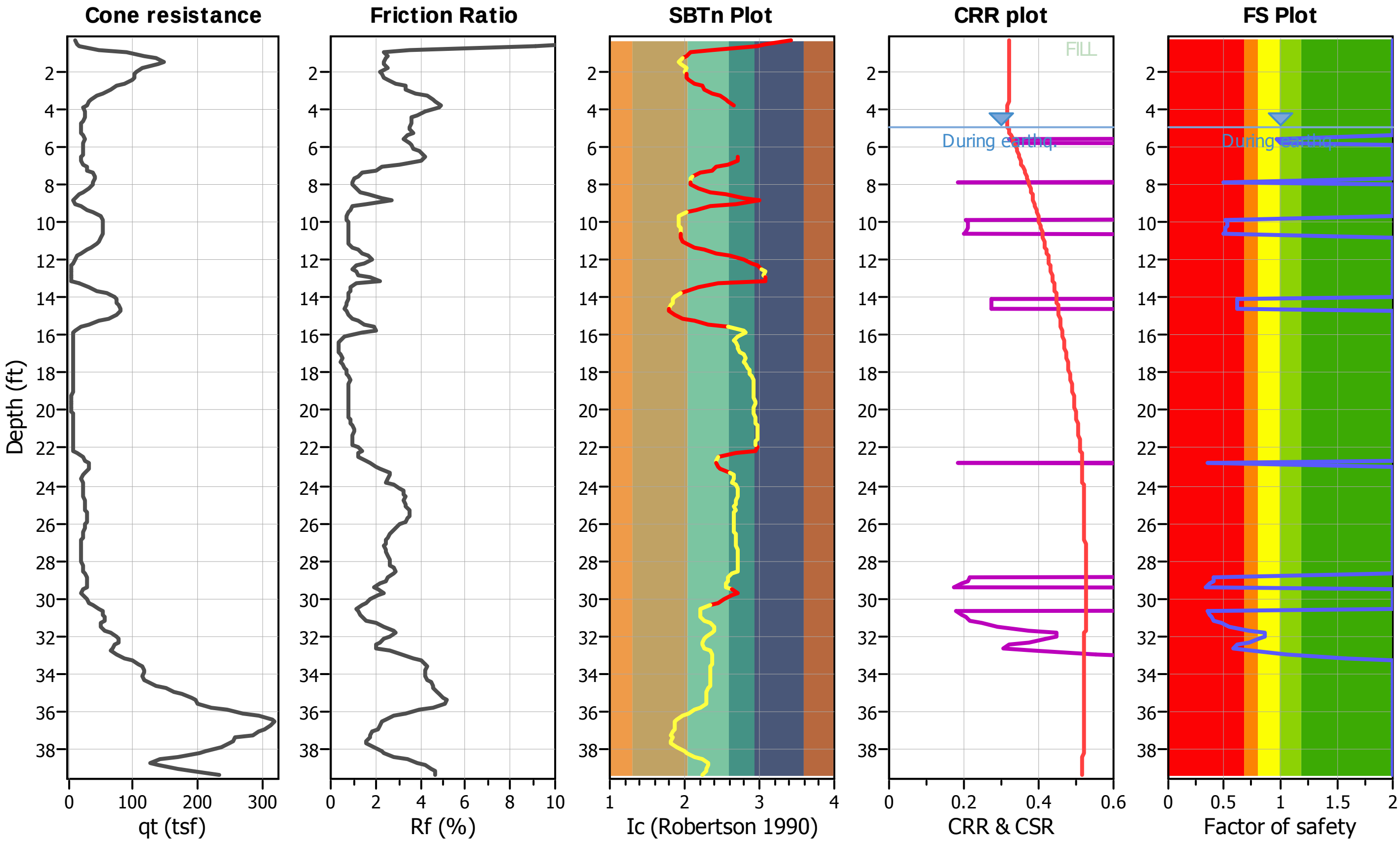
LIQUEFACTION ANALYSIS REPORT

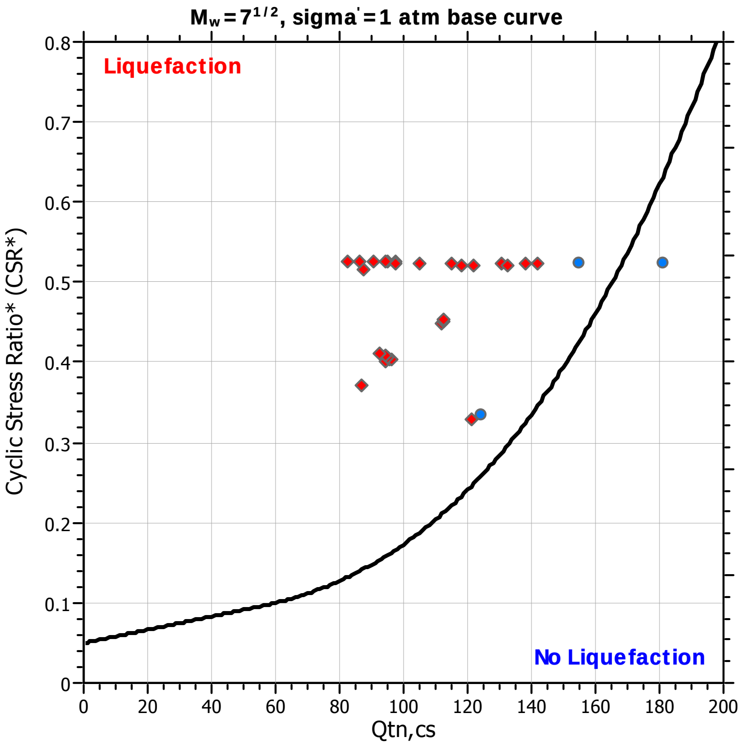
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

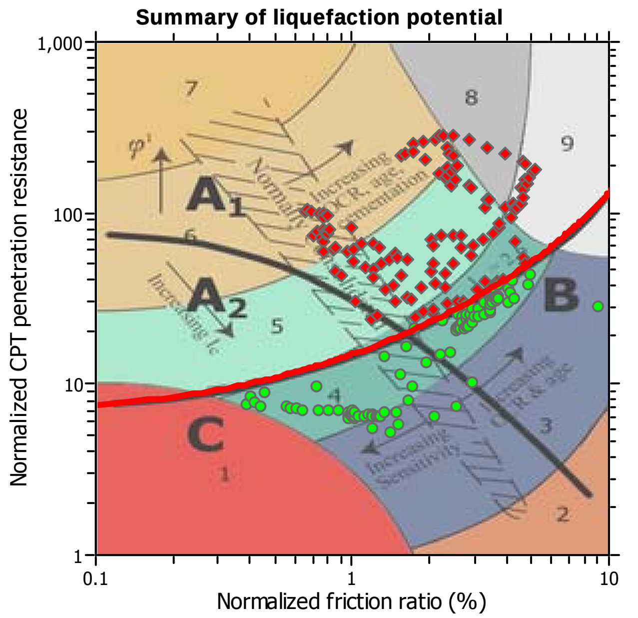
CPT file : 3-CPT03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

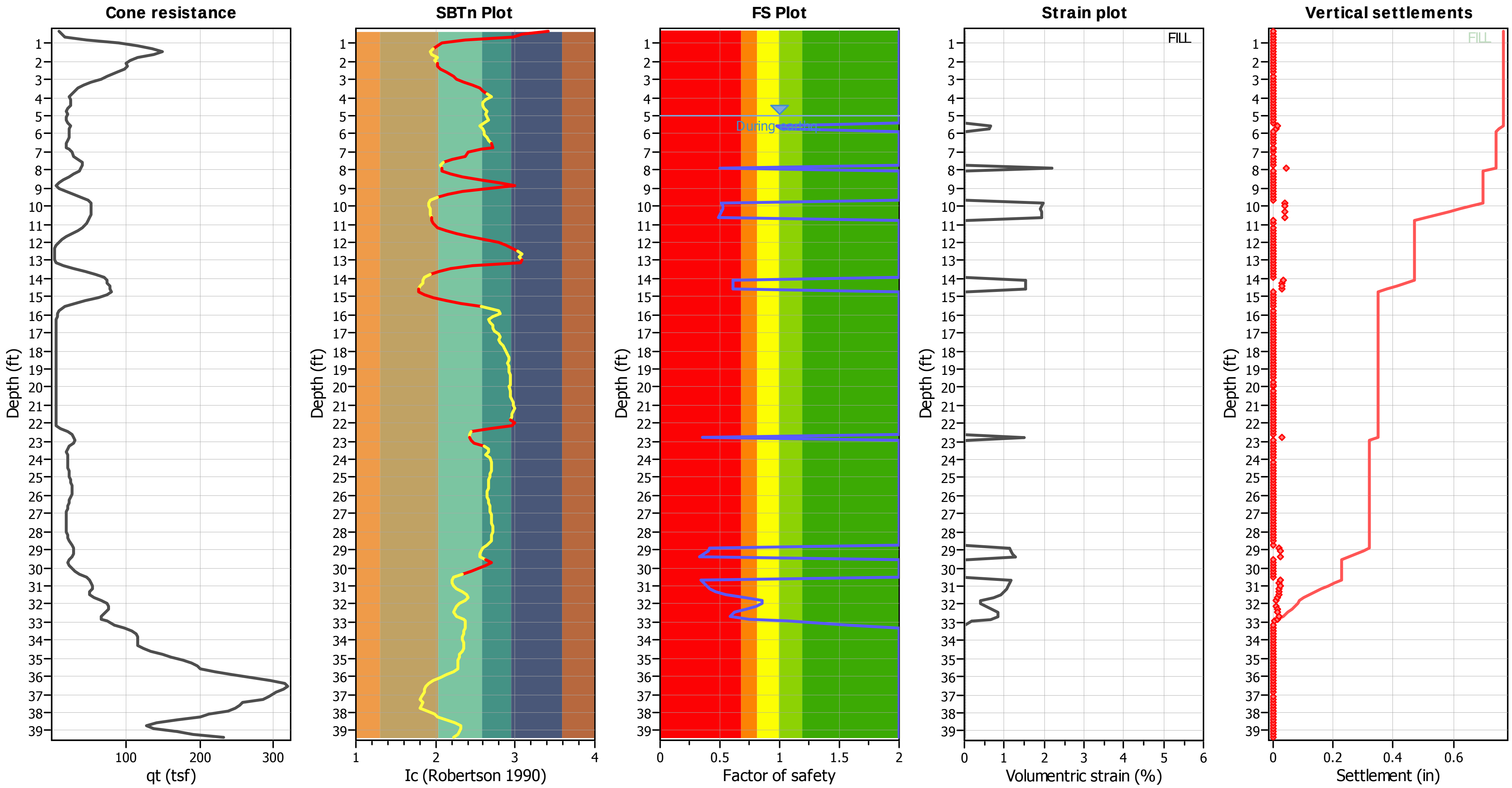






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction
Volumentric strain: Post-liquefaction volumentric strain

ENGEO
Expect Excellence

ENGEO

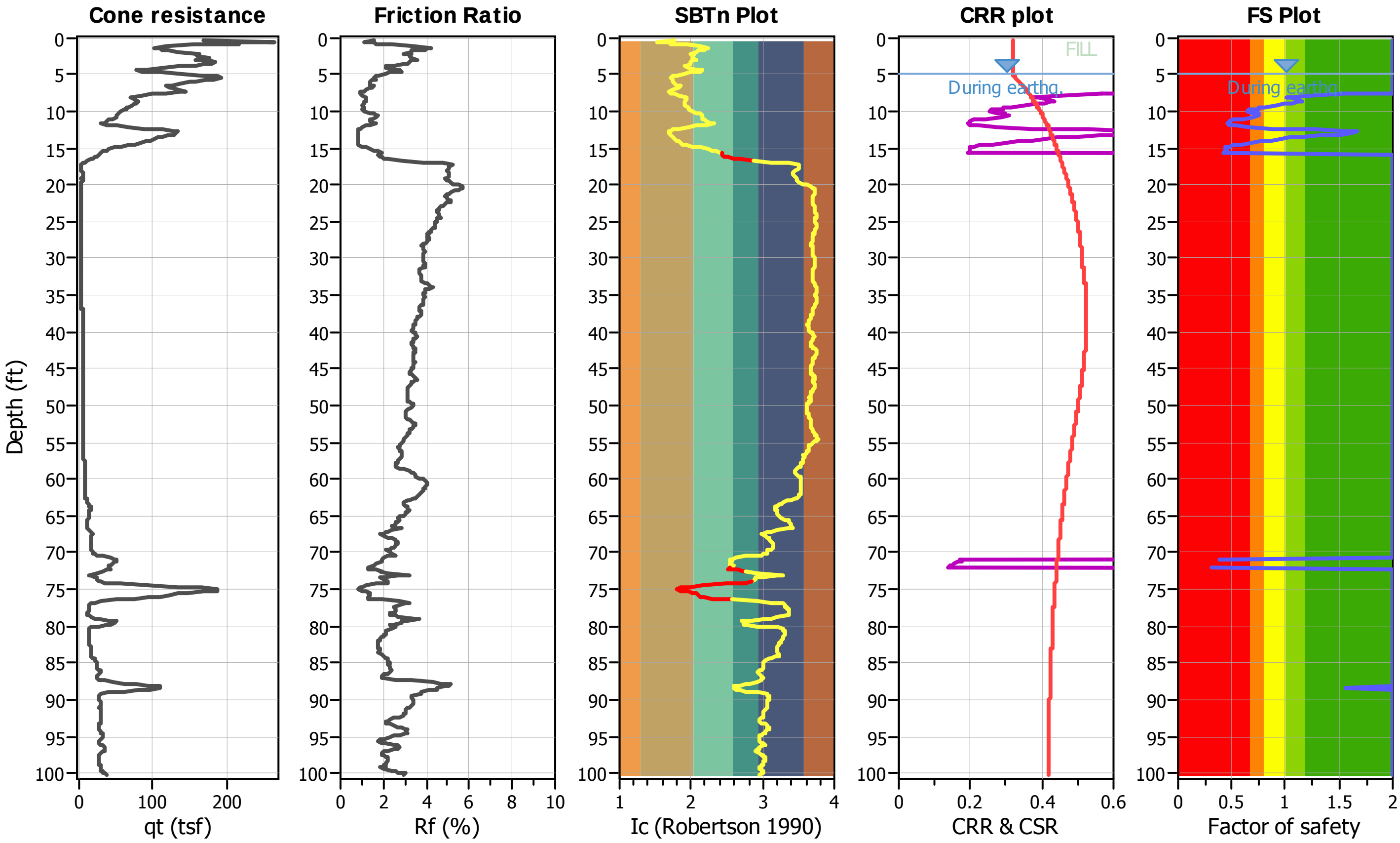
LIQUEFACTION ANALYSIS REPORT

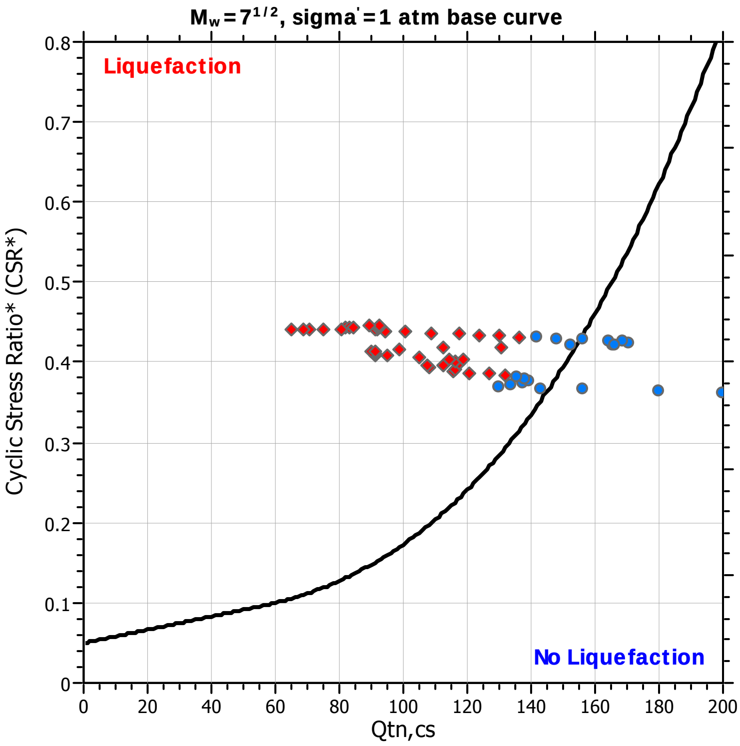
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

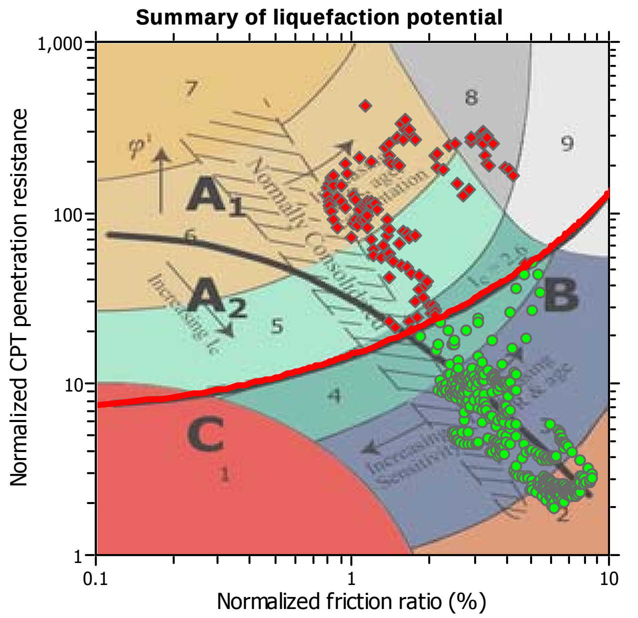
CPT file : 3-CPT04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

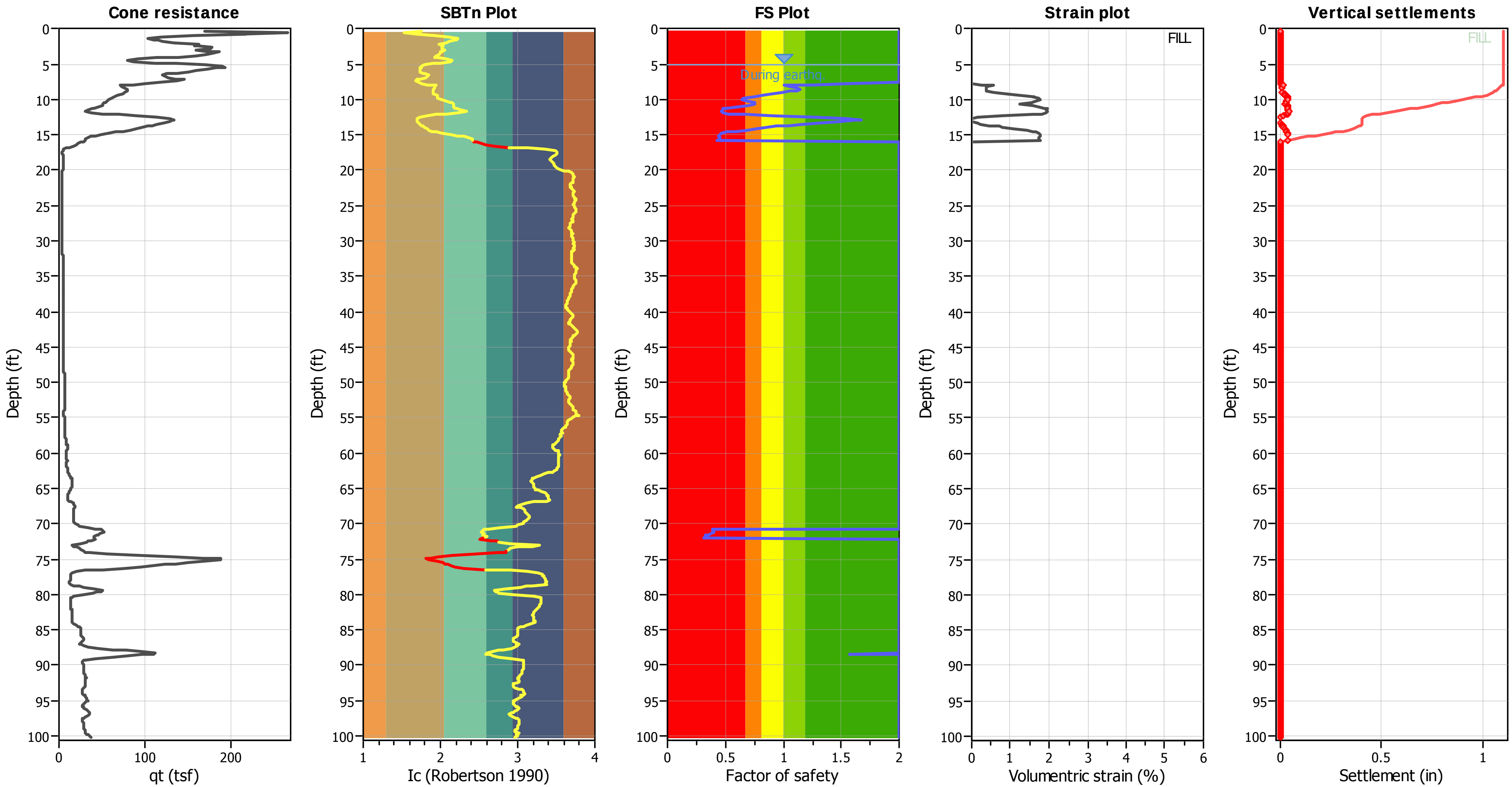






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

LIQUEFACTION ANALYSIS REPORT

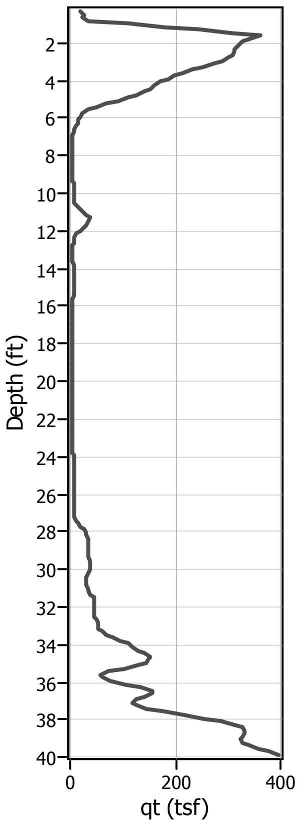
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

CPT file : 3-CPT05

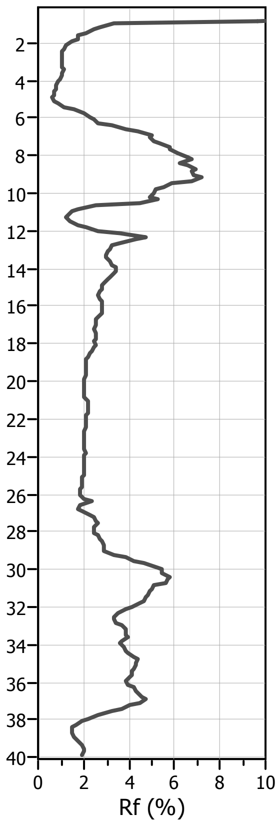
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

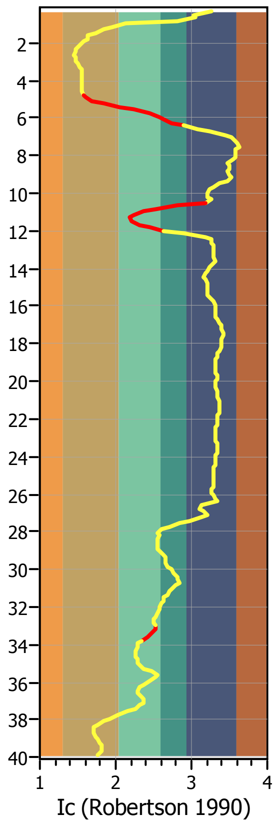
Cone resistance



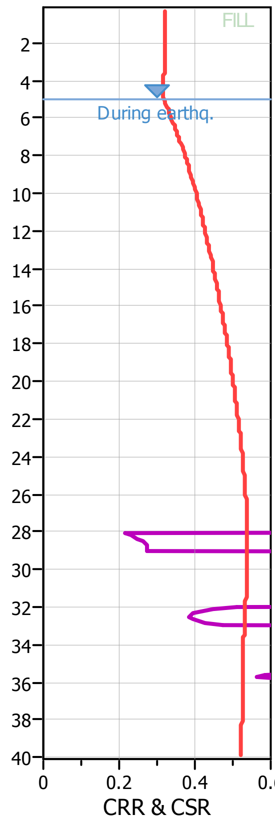
Friction Ratio



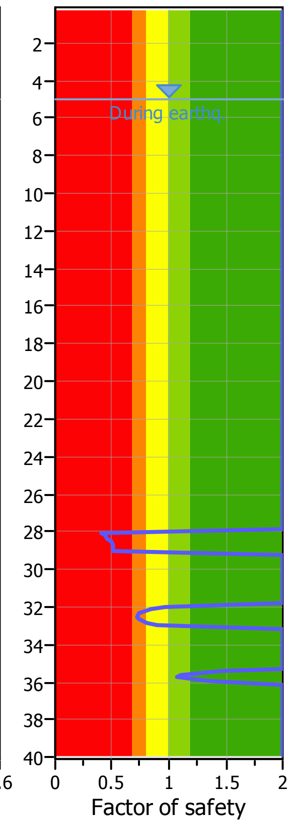
SBTn Plot



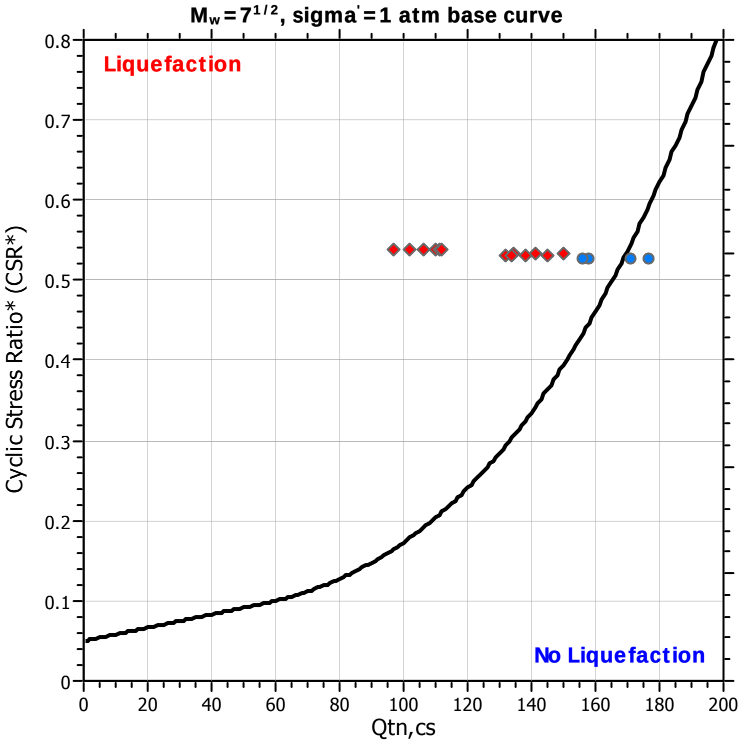
CRR plot



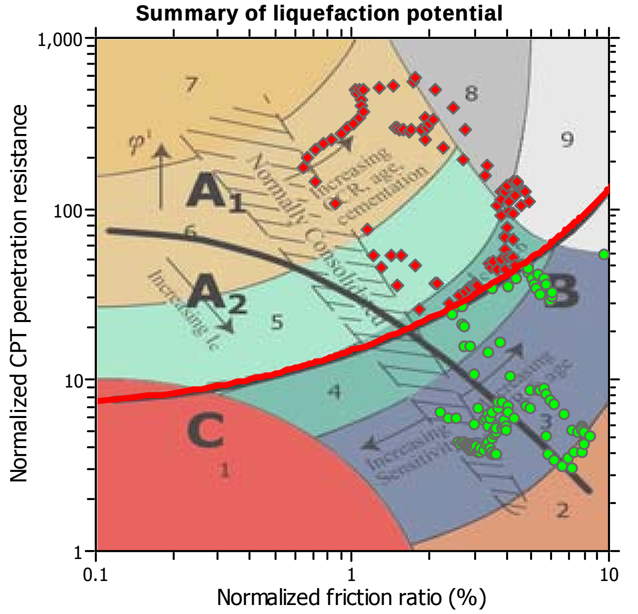
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

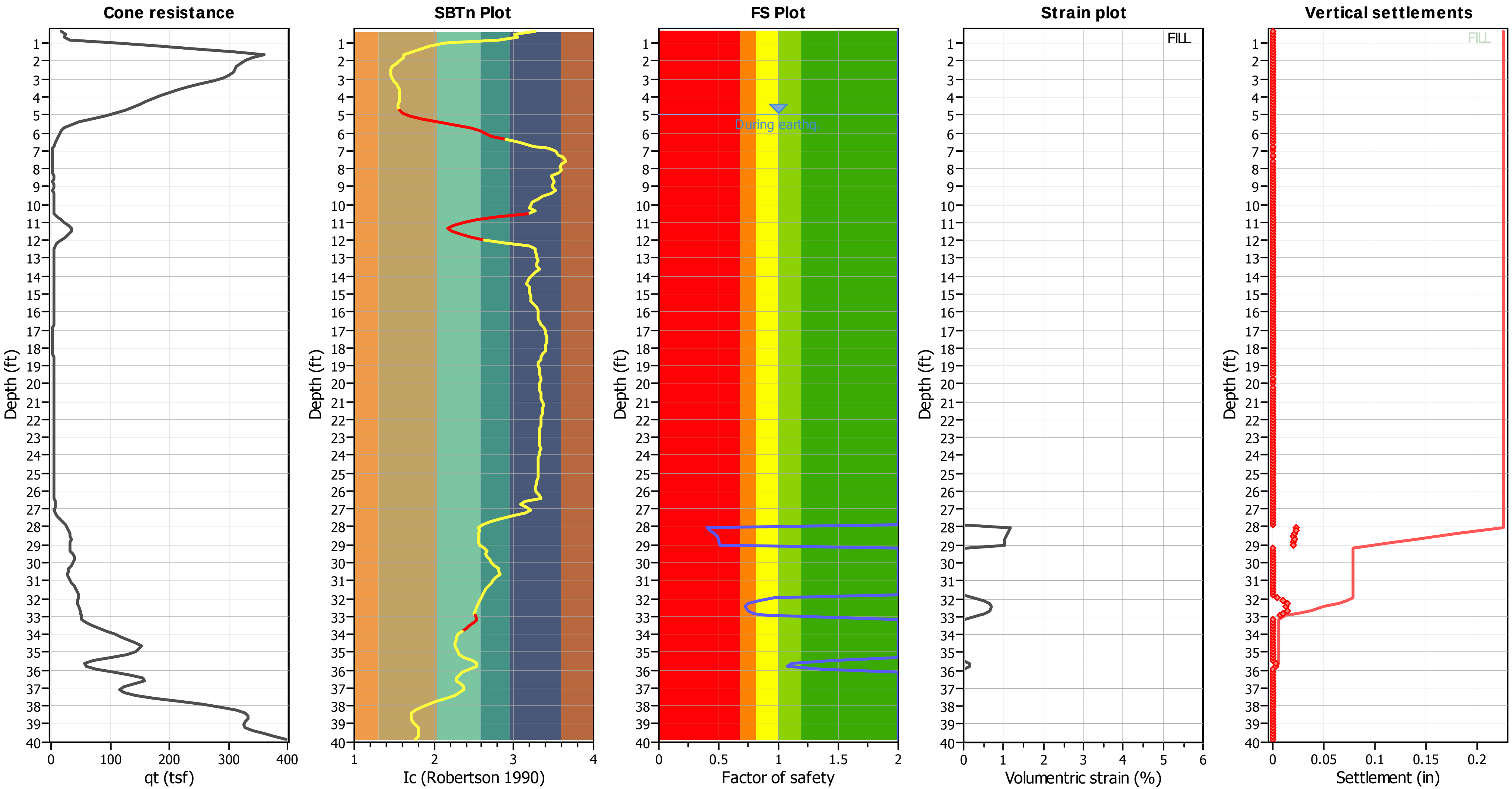


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

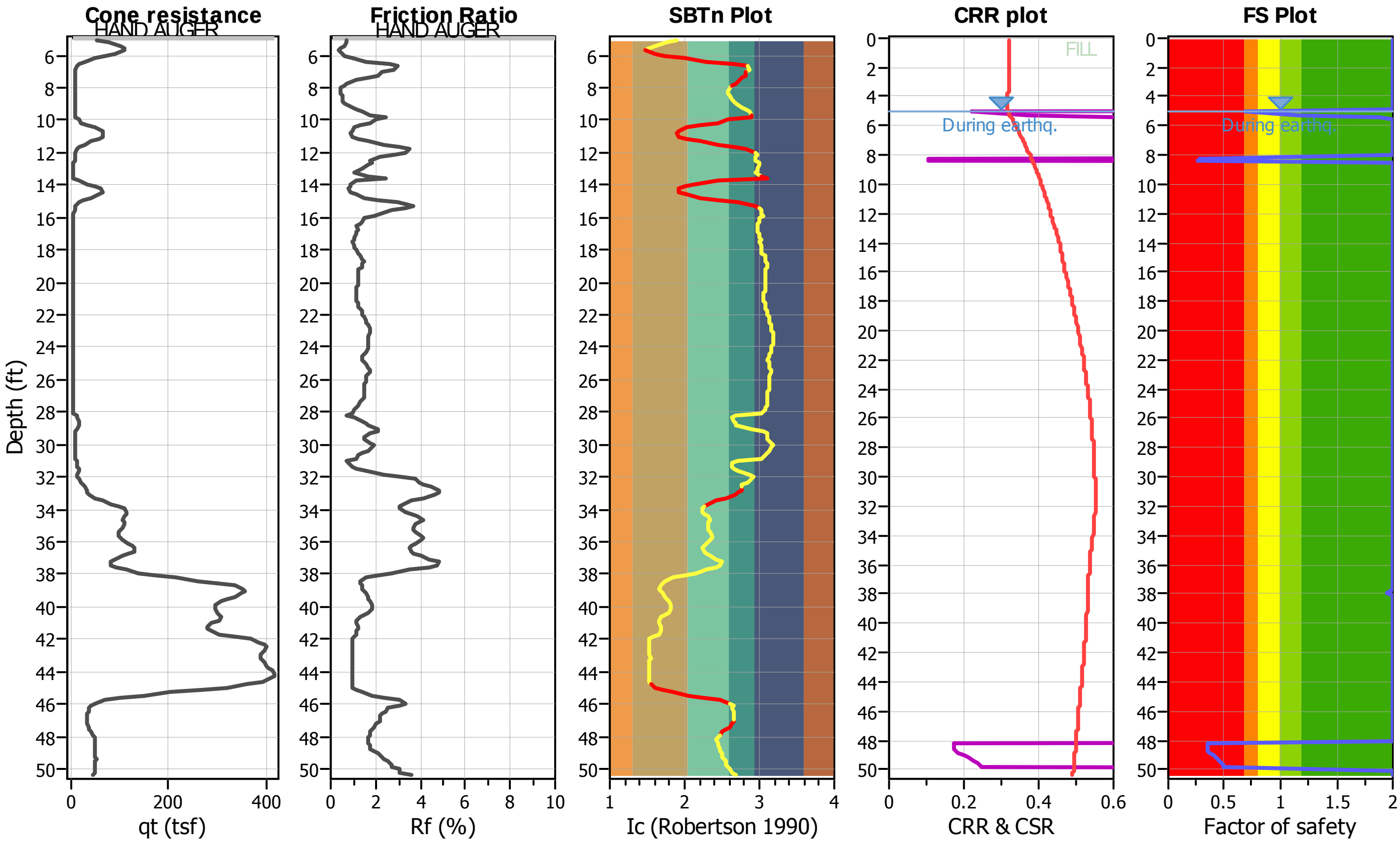
LIQUEFACTION ANALYSIS REPORT

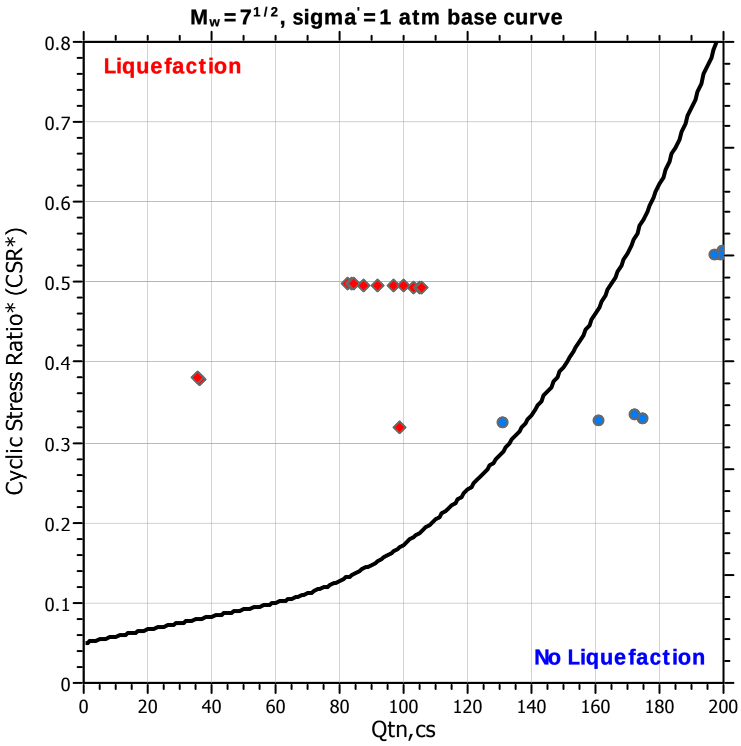
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

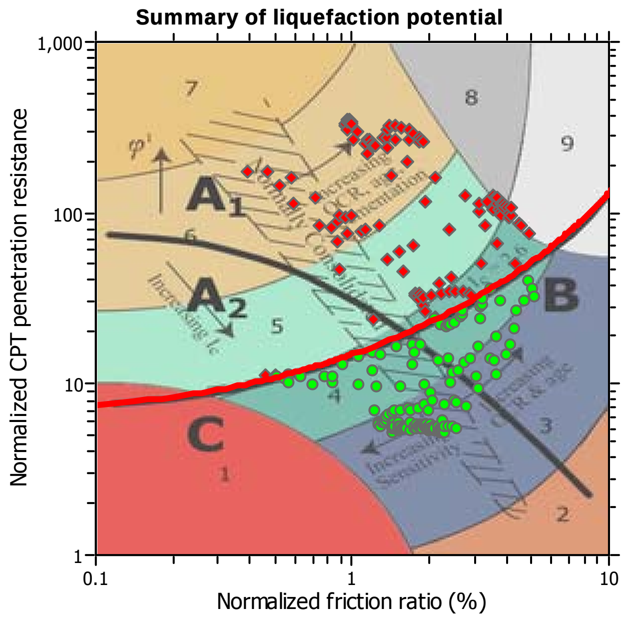
CPT file : 4-CPT01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

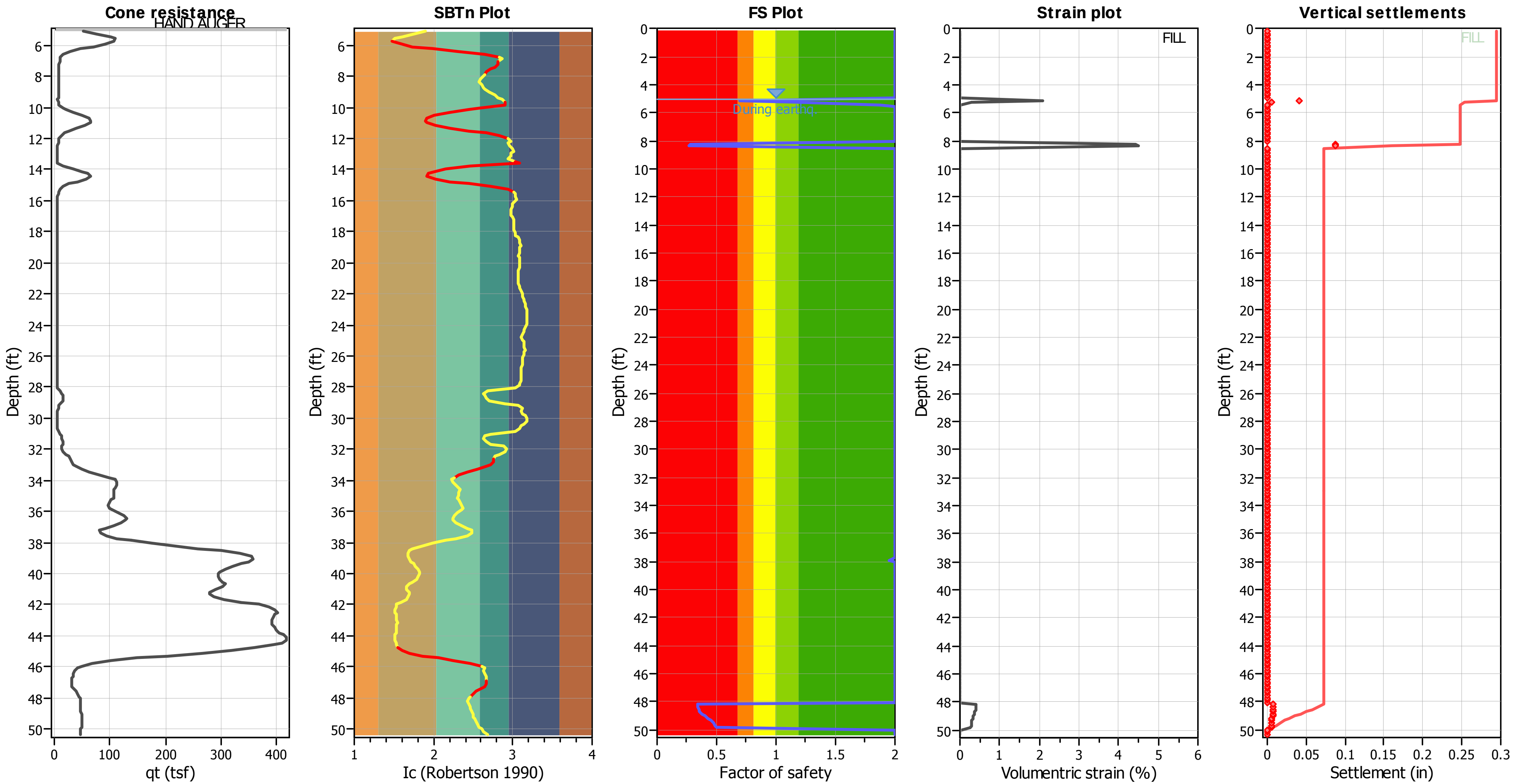






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

Expect Excellence

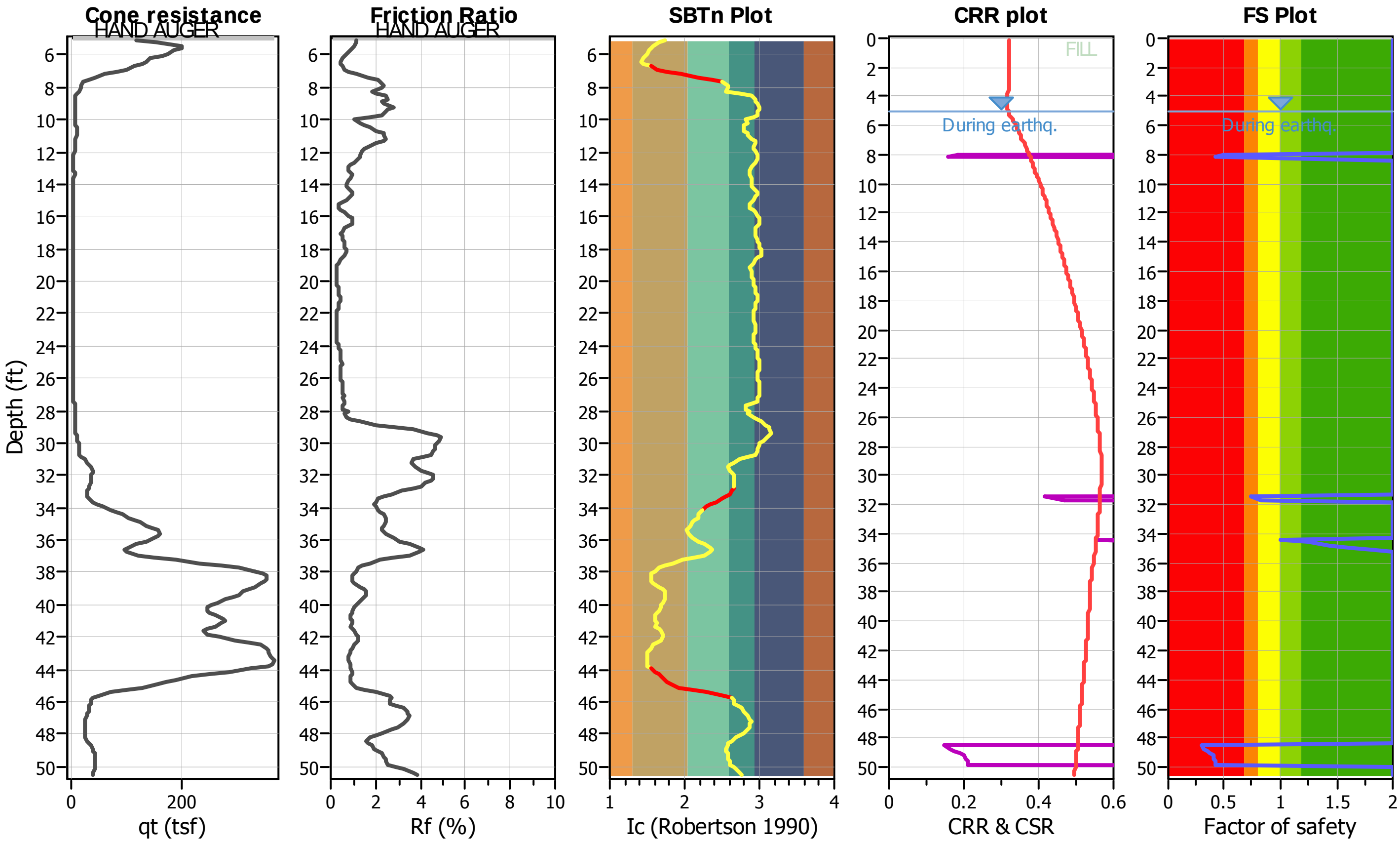
LIQUEFACTION ANALYSIS REPORT

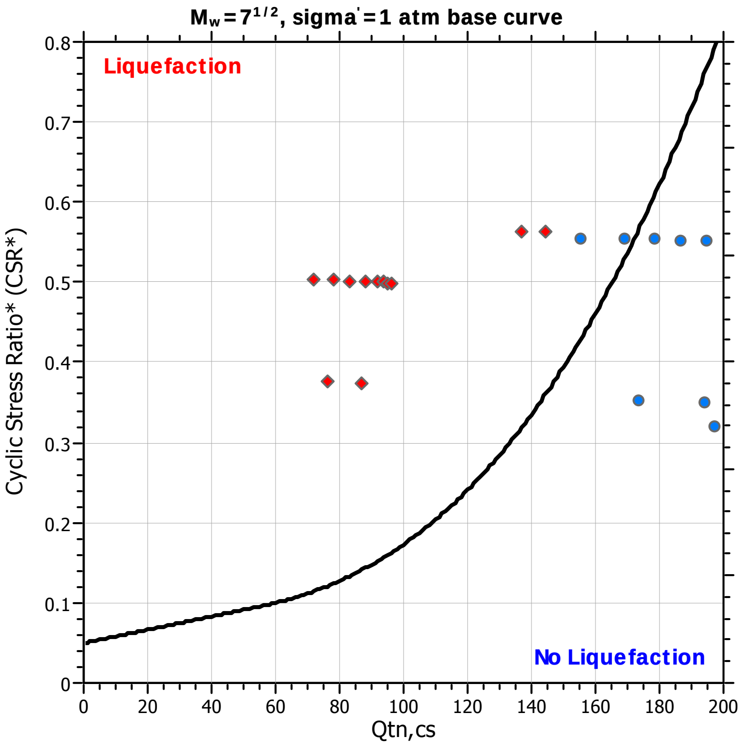
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

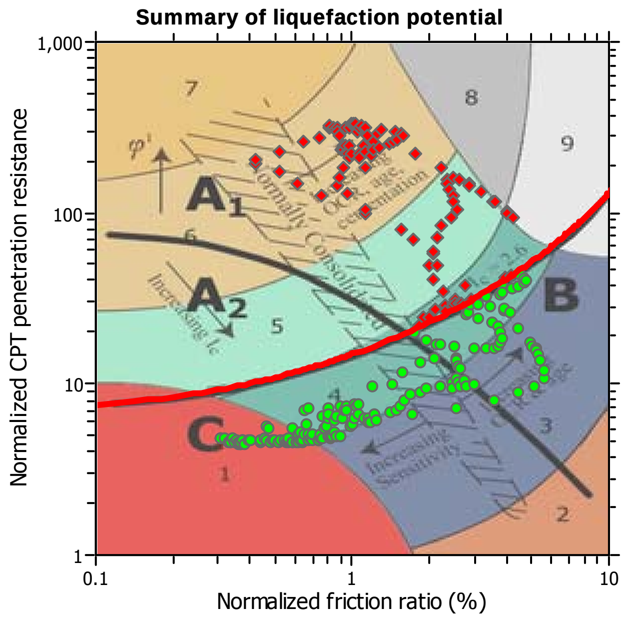
CPT file : 4-CPT02

Input parameters and analysis data

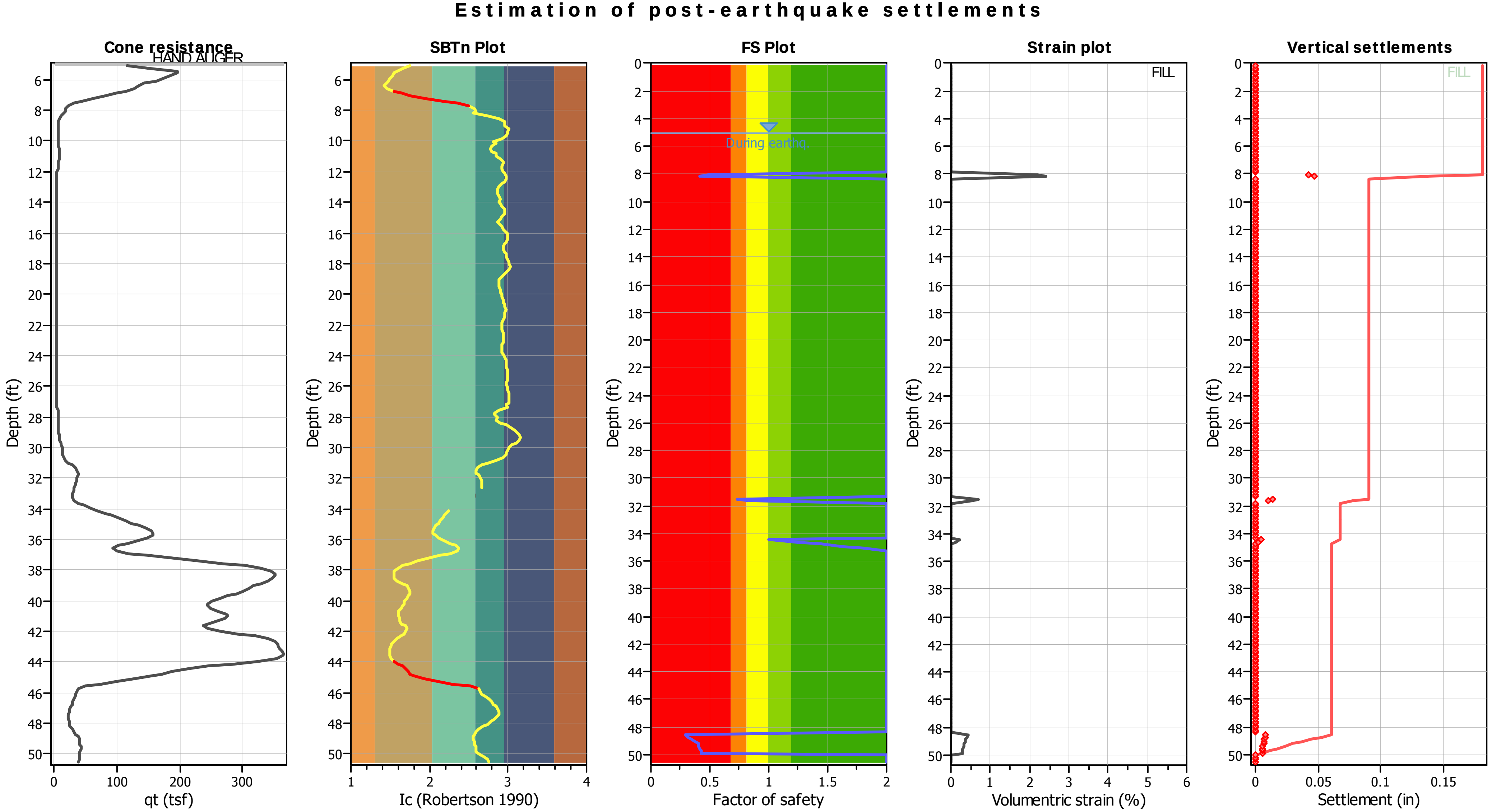
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO
Expect Excellence

ENGEO

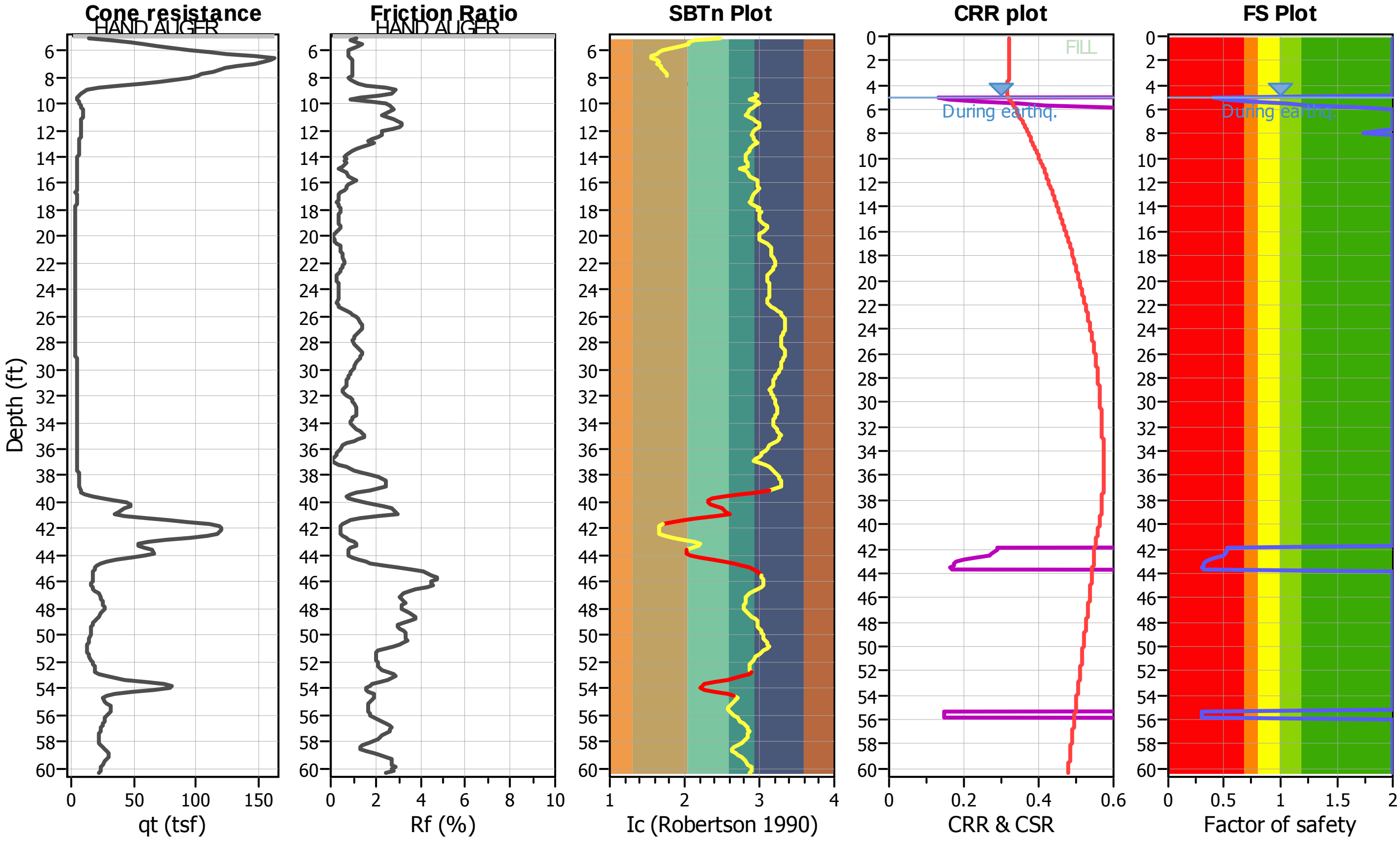
LIQUEFACTION ANALYSIS REPORT

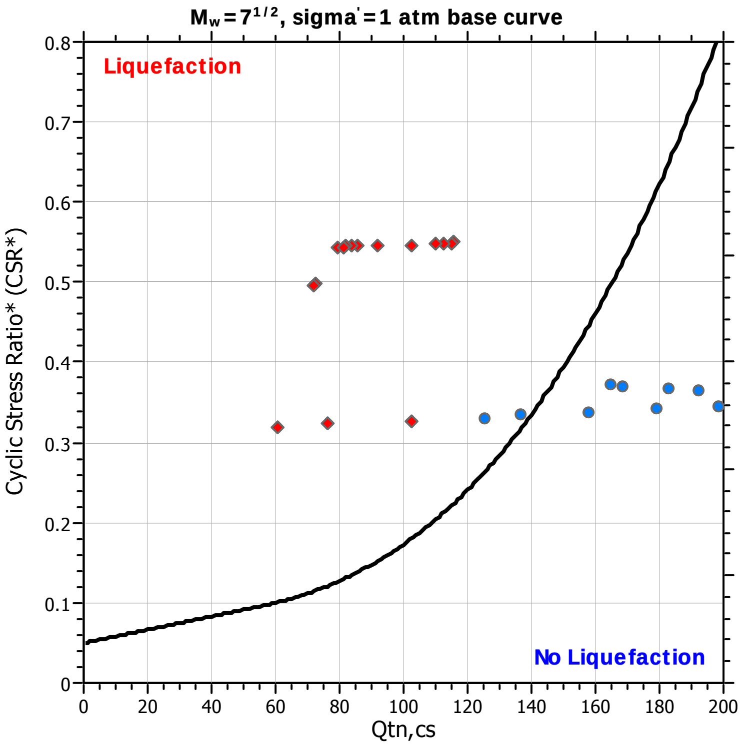
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

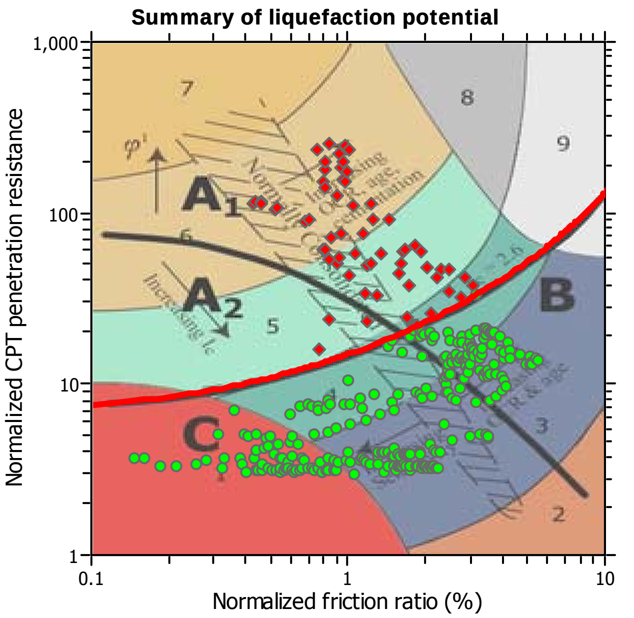
CPT file : 4-CPT03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft		
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

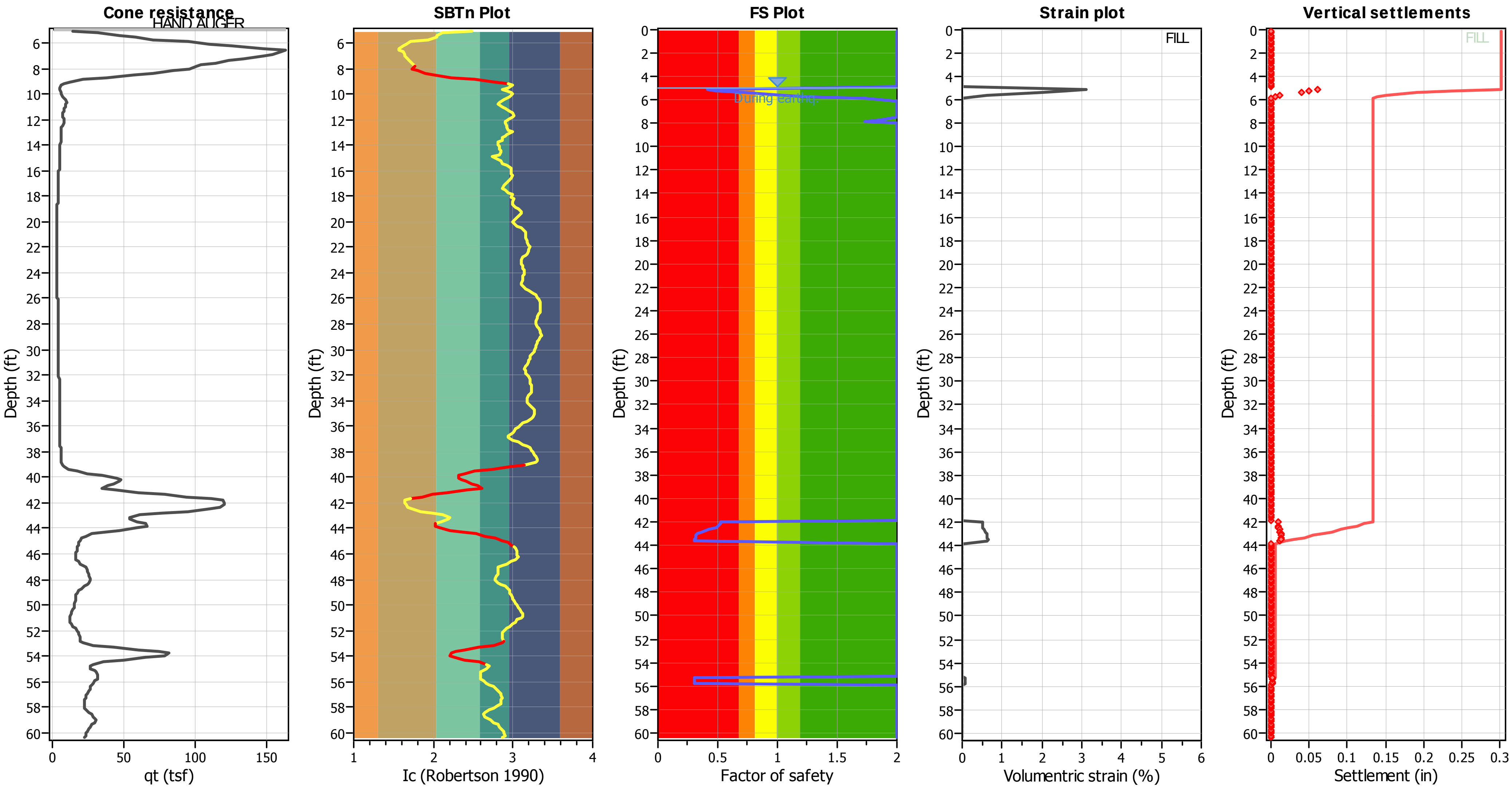






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO
Expect Excellence

ENGEO

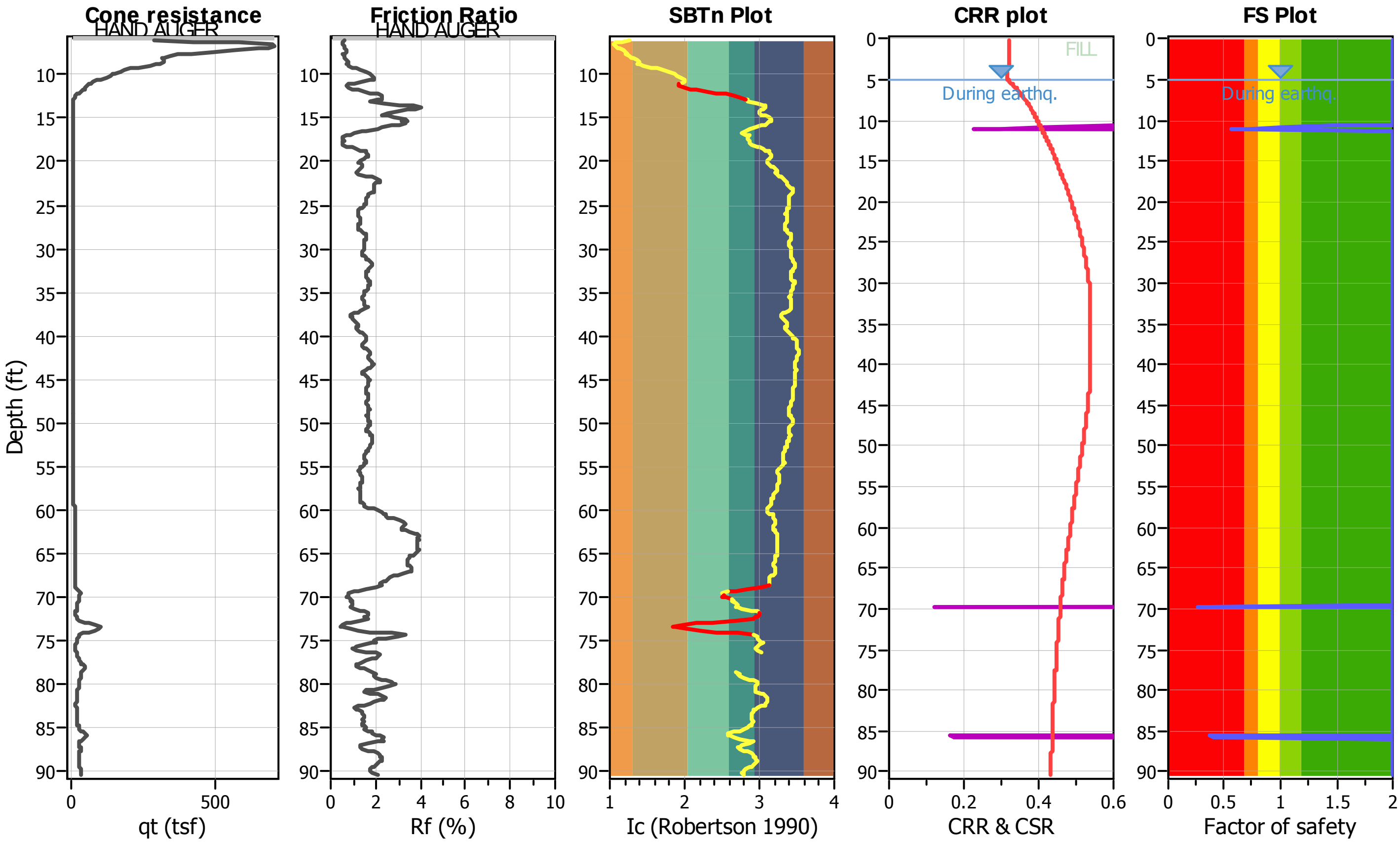
LIQUEFACTION ANALYSIS REPORT

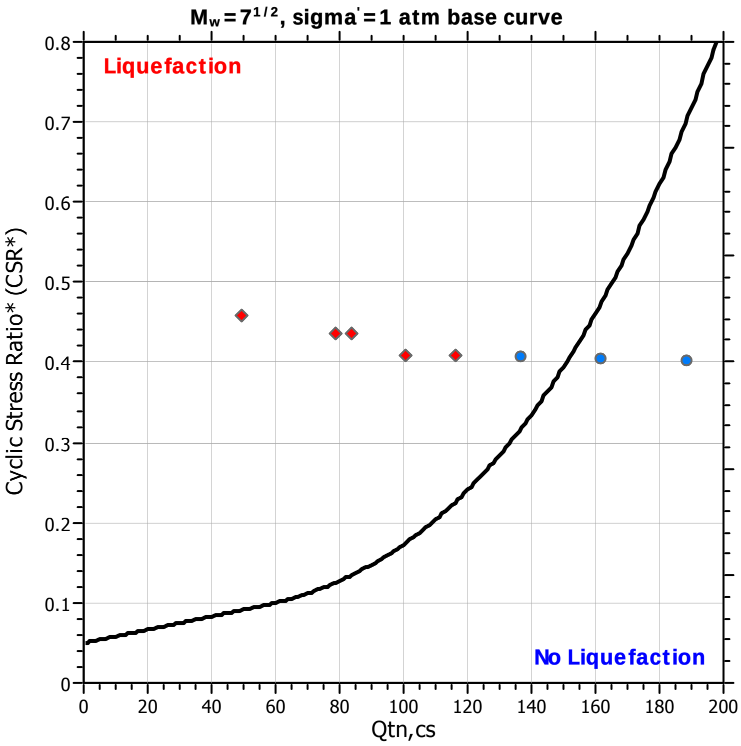
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

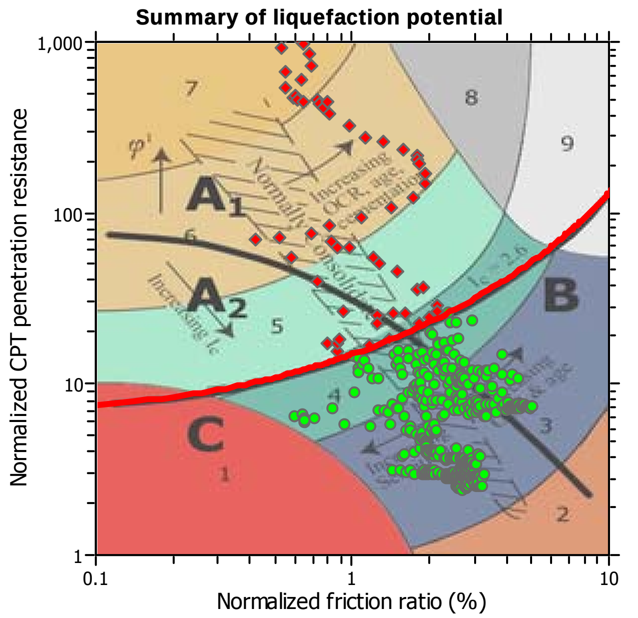
CPT file : 4-CPT04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

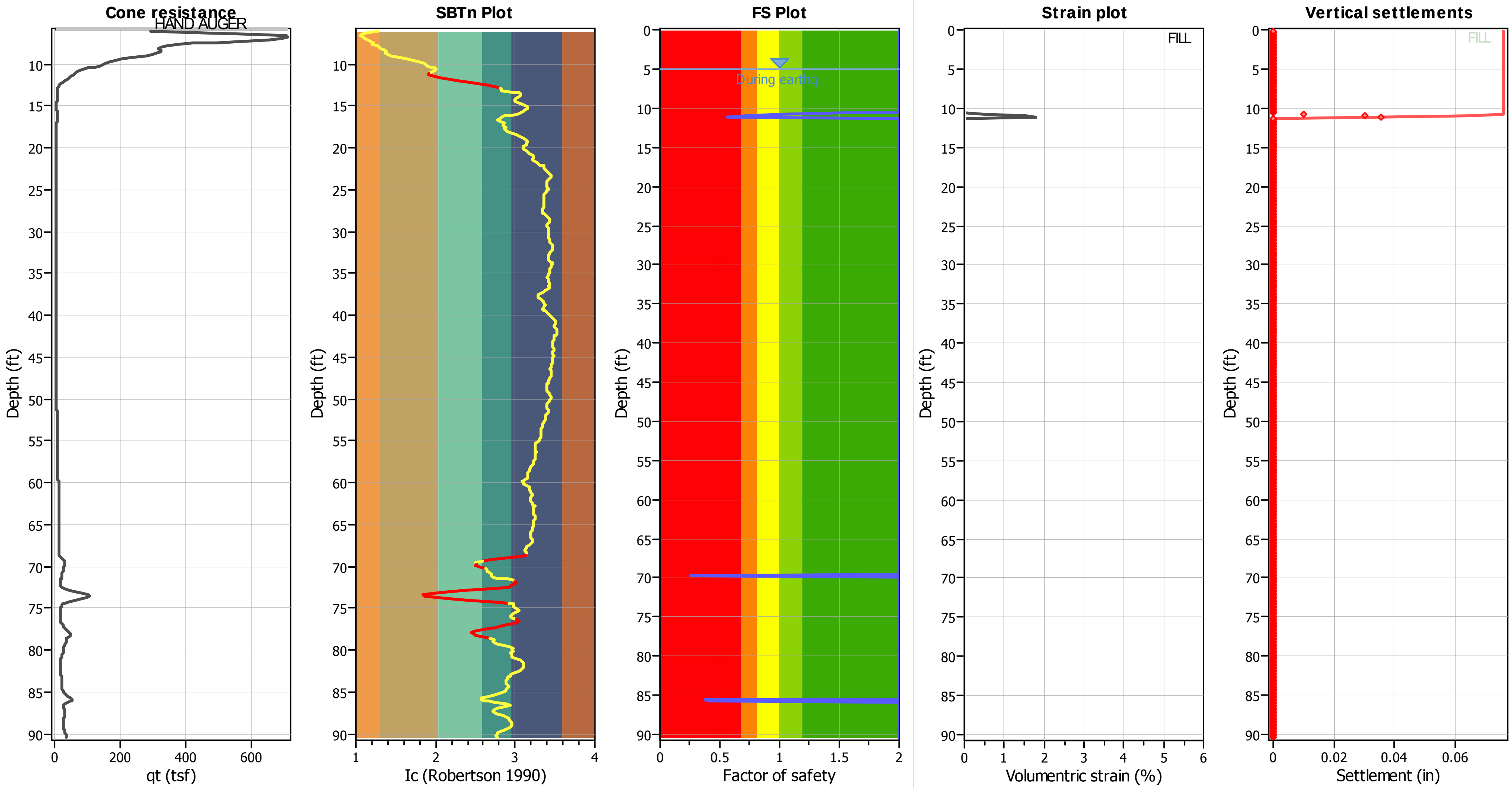






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO
Expect Excellence

ENGEO

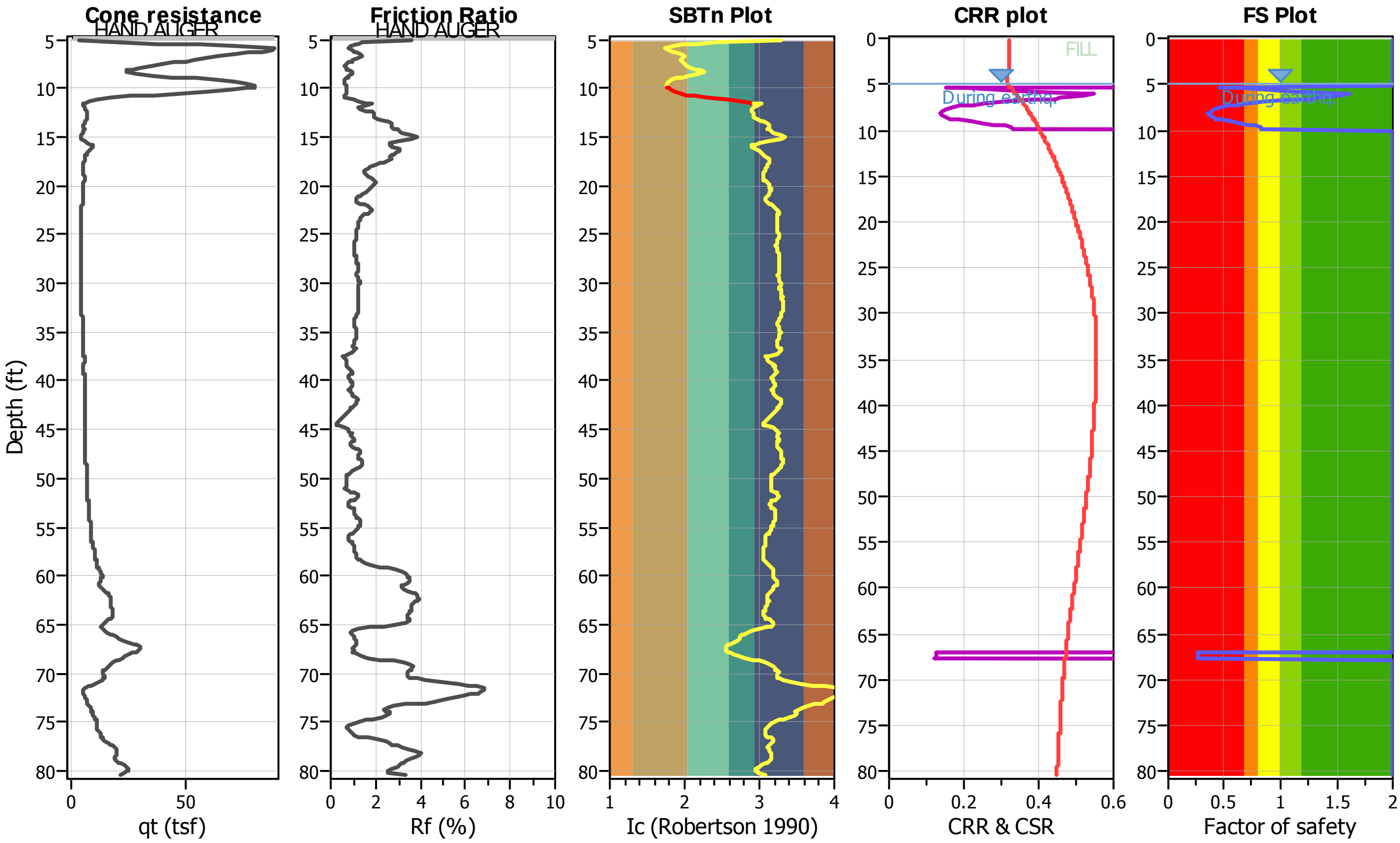
LIQUEFACTION ANALYSIS REPORT

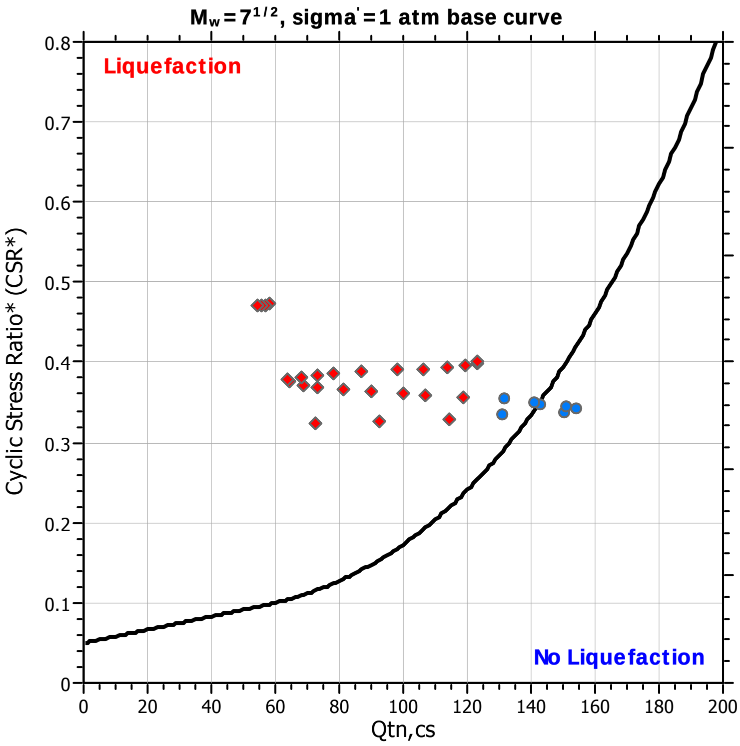
Project title : Encinal Terminals - 9769.000.000 **Location : Alameda, California**

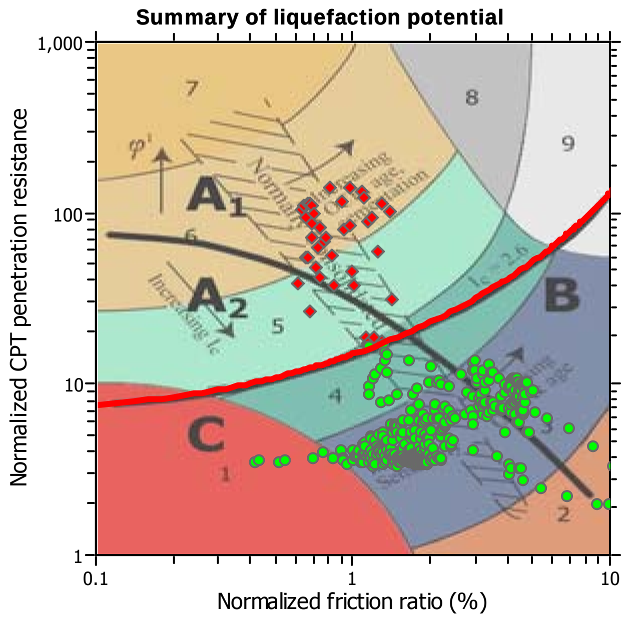
CPT file : 4-CPT05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.00 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	2.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	120.00 lb/ft^3	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.57	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

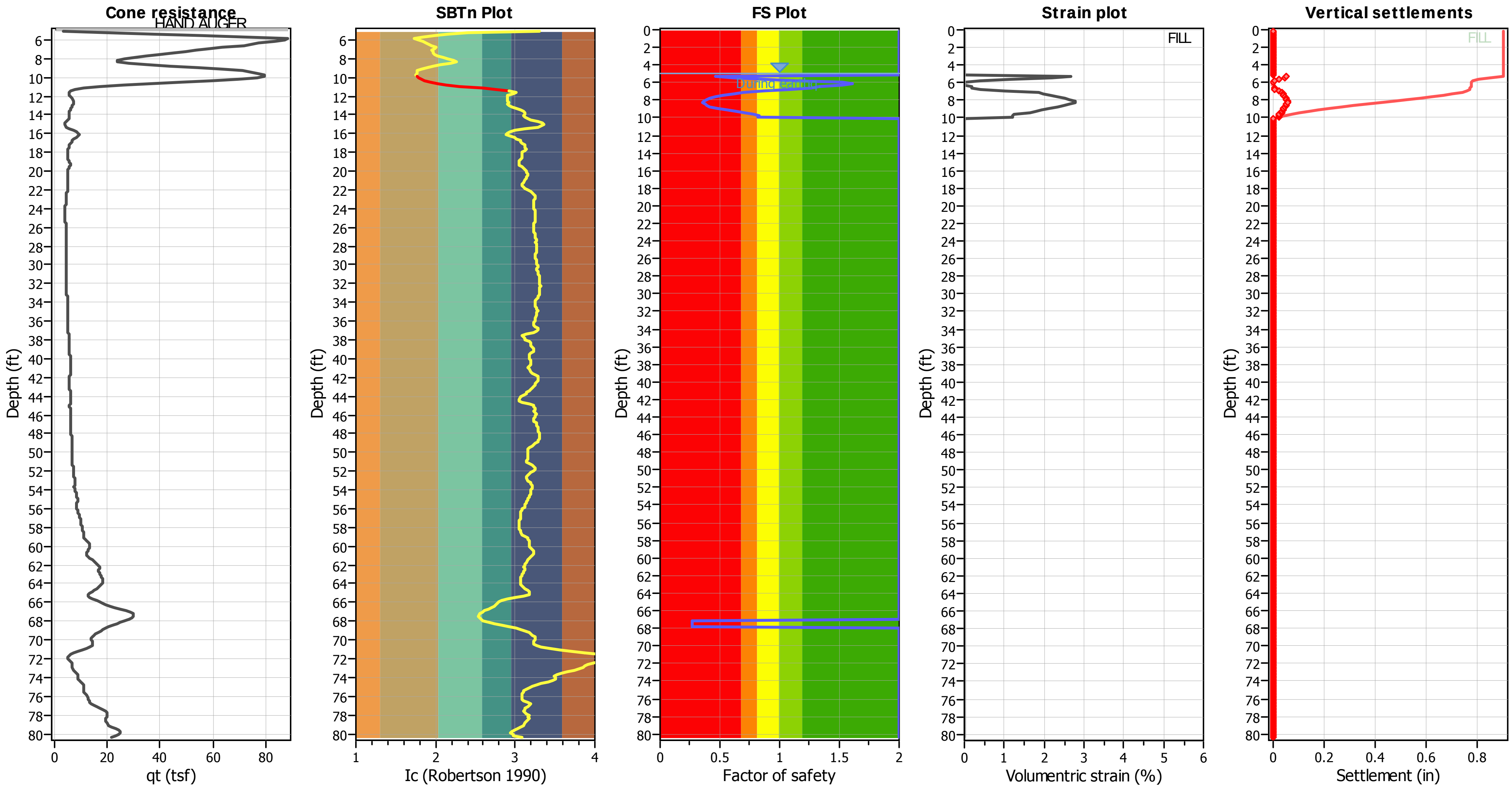






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

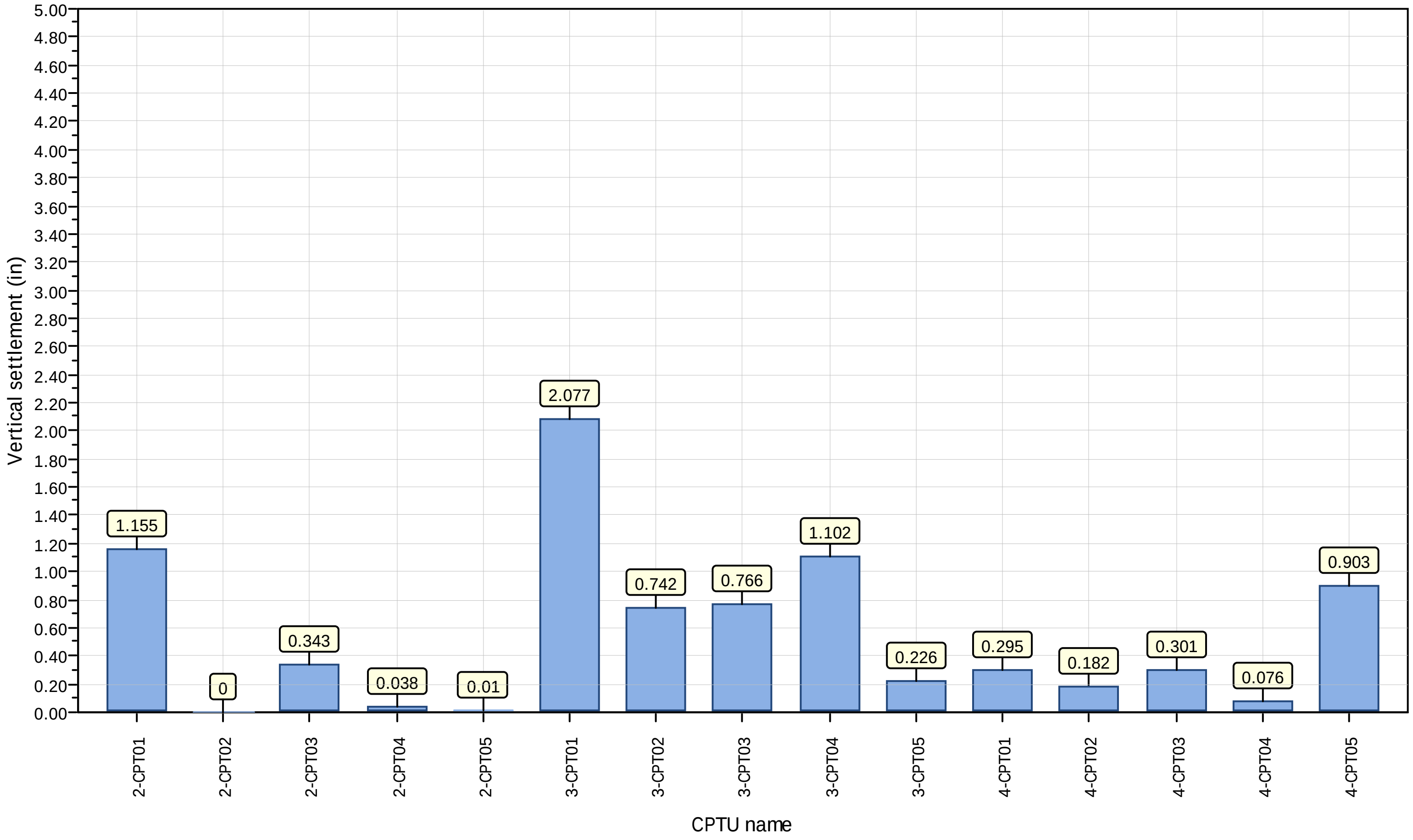
q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

ENGEO

Project title : Encinal Terminals - 9769.000.000

Location : Alameda, California

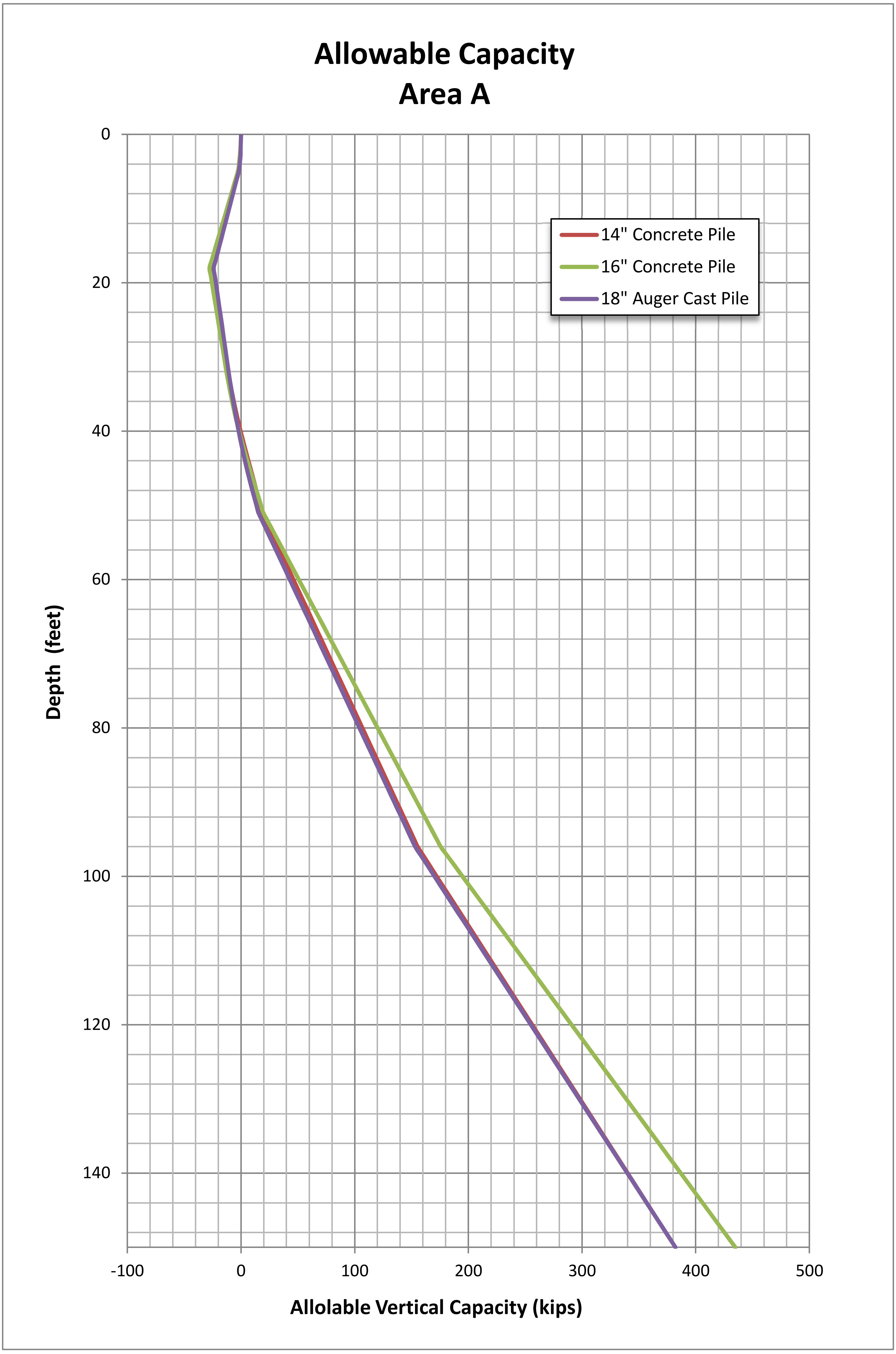
Overall vertical settlements report



APPENDIX D

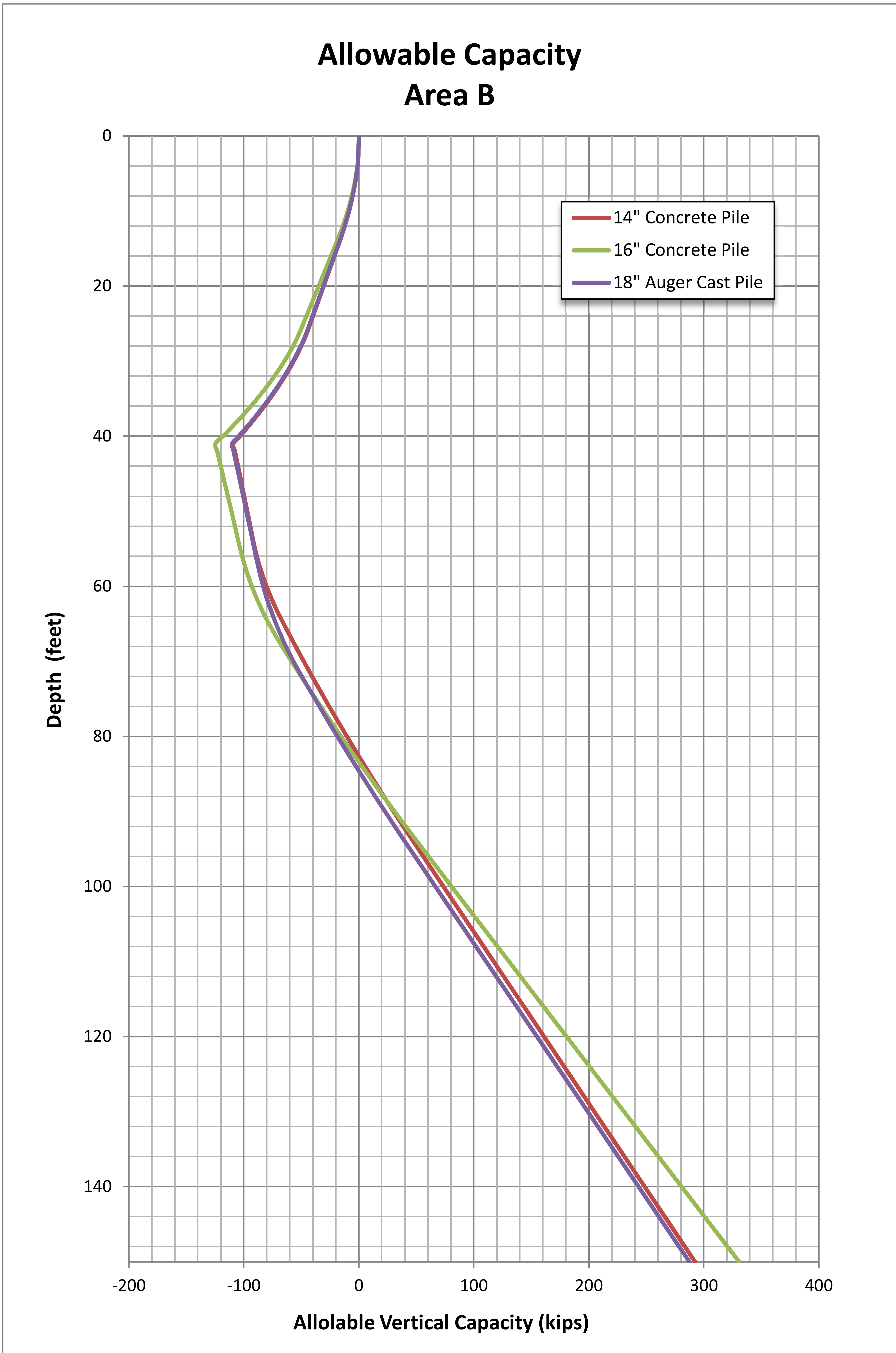
PRELIMINARY PILE CAPACITY CHARTS

APPENDIX D



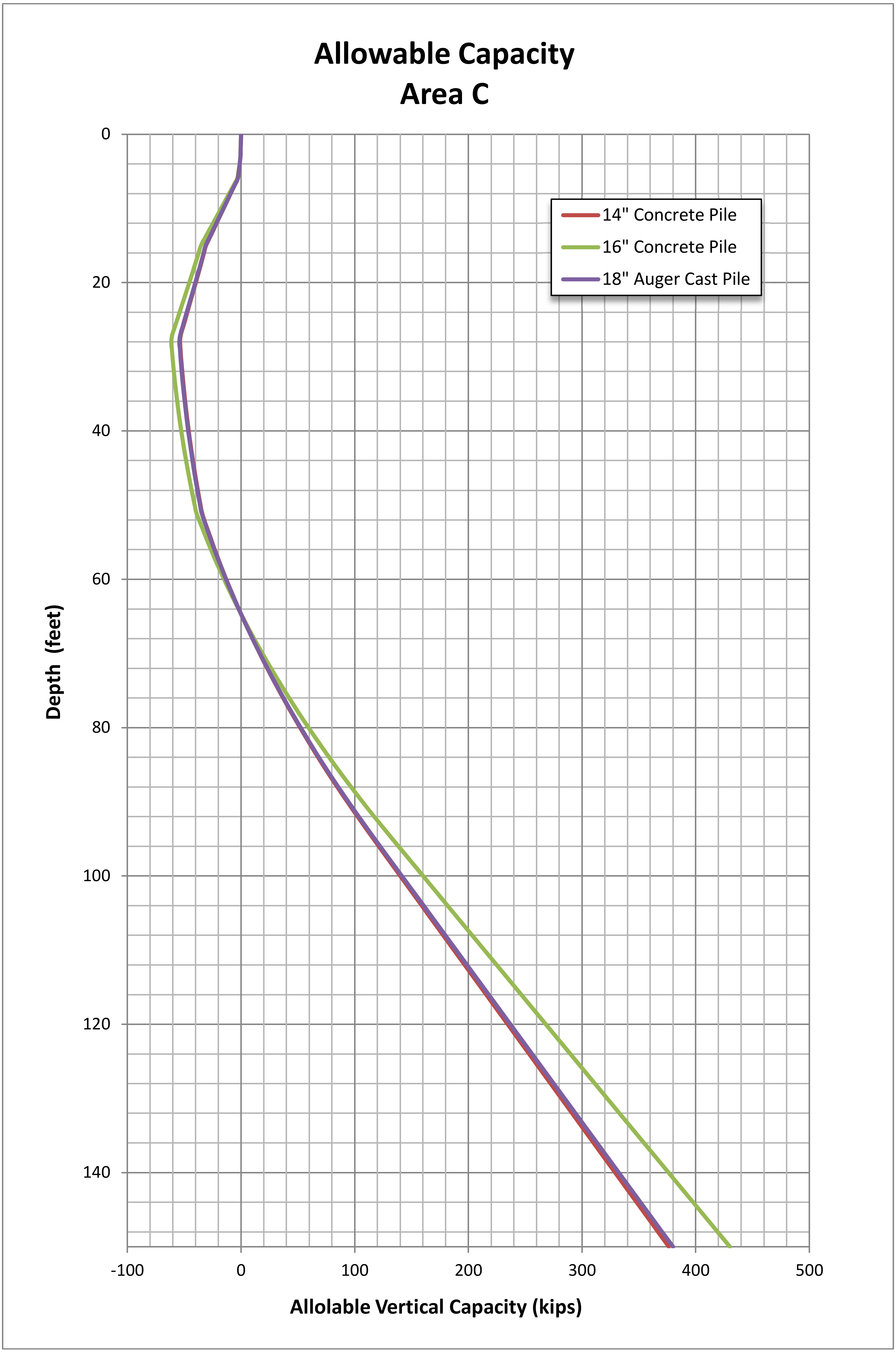
ENGEO
Expect Excellence

Encinal Terminal
9769.000.000
April 20, 2017

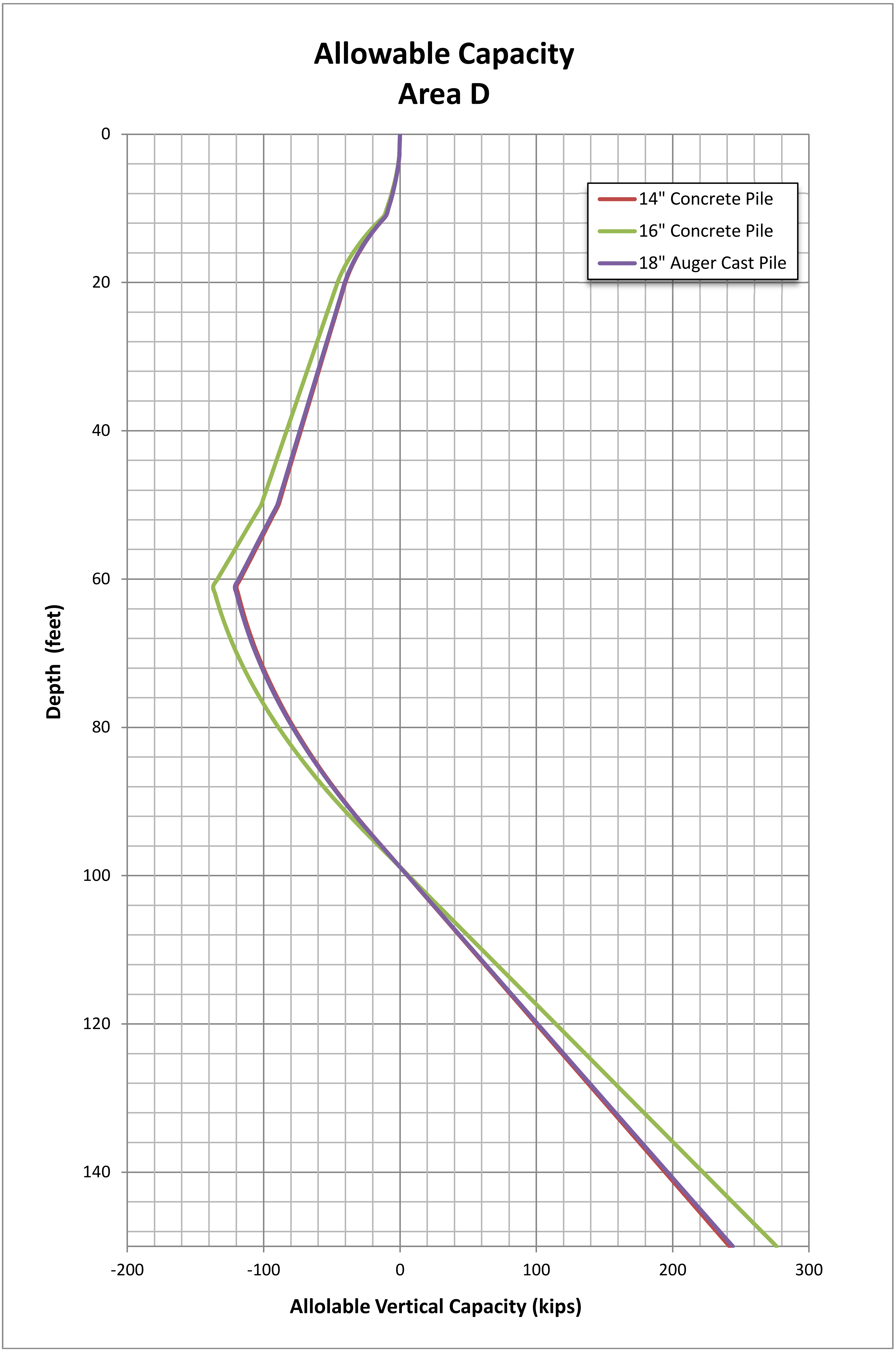


ENGEO
Expect Excellence

Encinal Terminal
9769.000.000
April 20, 2017



Encinal Terminal
9769.000.000
April 20, 2017



Encincal Terminal
9769.000.000
April 20, 2017